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GRAFTON POND DAM (NH. (U) CORPS OF ENGINEERS WALTHAM MA
NEW ENGLAND DIV NOV 78

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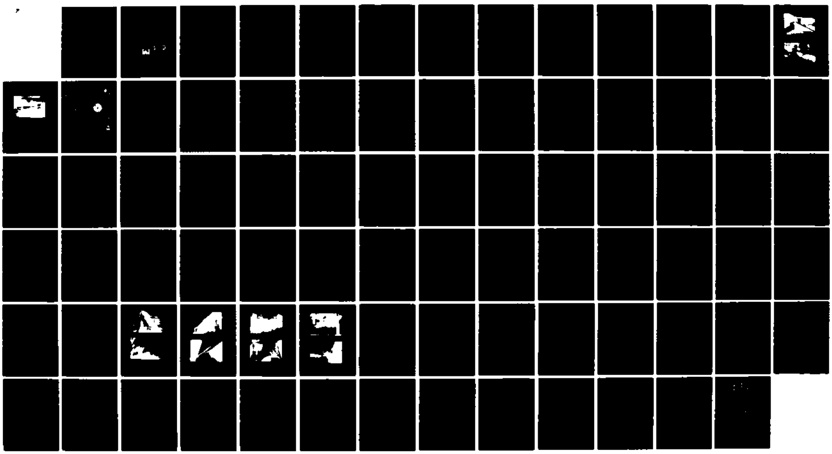

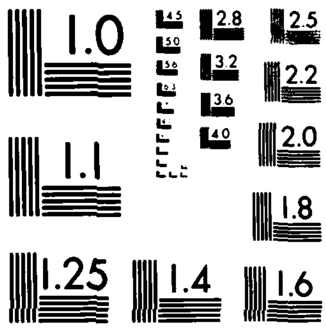

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

CONNECTICUT RIVER BASIN
GRAFTON, NEW HAMPSHIRE

GRAFTON POND DAM

NH 00119

NHWRB 96.01

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

NOVEMBER 1978

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER NH 00119	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Grafton Pond Dam NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE November 1978
		13. NUMBER OF PAGES 54
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) UNCLASSIFIED
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Connecticut Rsver Basin
Grafton, New Hampshire
Tributary to Bicknell Brook

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is a gravity concrete structure which about 285 ft. long and has a maximum height of 21 ft. It is intermediate in size with a low hazard potential. The dam is in very poor condition. Deterioration of the dam's two key structural elements, the upstram facing wall and its supporting buttresses, appears to seriously threaten the integrity of the structure. Due to the nature and extent of the deficiencies in this dam, the development of meaningful remedial measures of the dam should be implemented immediately

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:

NEDED

JAN 23 1979

Honorable Hugh J. Gallen
Governor of the State of New Hampshire
State House
Concord, New Hampshire 03301

Dear Governor Gallen:

I am forwarding to you a copy of the Grafton Pond Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Water Resources Board, the cooperating agency for the State of New Hampshire. In addition, a copy of the report has also been furnished the owner, New Hampshire Water Resources Board, 37 Pleasant Street, Concord, New Hampshire 03301, ATTN: Mr. George M. McGee, Sr., Chairman.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Water Resources Board for your cooperation in carrying out this program.

Sincerely yours,

JOHN P. CHANDLER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

GRAFTON POND DAM
NH 00119

CONNECTICUT RIVER BASIN
GRAFTON, NEW HAMPSHIRE

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

NATIONAL DAM INSPECTION PROGRAM

PHASE I REPORT

Identification No.:	NH 00119
NHWRB No.:	96.01
Name of Dam:	GRAFTON POND DAM
Town:	Grafton
County and State:	Grafton, New Hampshire
Stream:	Tributary to Bicknell Brook
Date of Inspection:	September 21, 1978

BRIEF ASSESSMENT

The Grafton Pond Dam is an Ambursen dam, a gravity concrete structure named after its designer. The dam is approximately 285 feet long and has a maximum height of 21 feet. Discharges from the reservoir are via a 24 foot wide concrete ogee spillway or two 2 feet wide by 3 feet high steel sluice gates. The dam, which is owned by the New Hampshire Water Resources Board (NHWRB), was built in 1918.

Grafton Pond receives runoff from 3.6 square miles of moderately sloping, heavily forested terrain. The dam's maximum impoundment of approximately 3000 acre-feet places it in the INTERMEDIATE size category, while the lack of any downstream hazard for a considerable distance below the dam results in a LOW hazard potential classification.

Based on the size and hazard potential ratings and in accordance with the Corp's guidelines, the Test Flood (TF) is between the 100 year flood and one half the Probable Maximum Flood (PMF). The selected TF inflow of 1800 cfs results in a discharge at the dam of approximately 900 cfs. Since the discharge capacity of the structure at maximum pond elevation is only approximately 622 cfs, the TF overtops the dam by approximately 0.4 feet.

The Grafton Pond Dam is in VERY POOR condition at the present time. Deterioration of the dam's two key structural elements, the upstream facing wall and its supporting buttresses, appears to seriously threaten the integrity of the structure. For this reason, it is recommended that immediately upon receipt of this report the pond be lowered to at least 7 feet below the spillway crest and that a qualified structural engineer be retained to conduct a detailed inspection and analysis of the dam under fully drawn down conditions.

Until such an inspection can be arranged, it is further recommended that the impoundment be maintained below gage elevation 9.0, or 7 feet below the spillway.

Due to the nature and extent of the deficiencies in this dam, the development of meaningful remedial measures for this structural problem is beyond the scope of this Phase I investigation and should be deferred pending completion of the structural inspection. The only non-structural remedial measure involves the training of local officials in dam operations to decrease response time in operating the dam in the event of an emergency.

The structural investigation recommended above should be accomplished immediately and the dam should remain drawn down until such time as a permanent solution is developed. Based on the dam's VERY POOR condition, periodic inspections should be conducted every 6 months.

THE STATE OF NEW HAMPSHIRE
WILLIAM S. ZOINO
No. 3226
REGISTERED PROFESSIONAL ENGINEER

REGISTERED PROFESSIONAL ENGINEER
NICHOLAS ANTHONY CAMPAGNA, JR.
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CIVIL
STATE OF CALIFORNIA

William S Zoino

Nicholas A. Campagna Jr

William S. Zoino
New Hampshire Registration 3226

Nicholas A. Campagna, Jr.
California Registration 21006

This Phase I Inspection Report on Grafton Pond Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Richard F. Doherty

RICHARD F. DOHERTY, MEMBER
Water Control Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

Joseph A. McElroy

JOSEPH A. MCELROY, CHAIRMAN
Chief, NED Materials Testing Lab.
Foundations & Materials Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the Test Flood should not be interpreted as necessarily posing a highly inadequate condition. The Test Flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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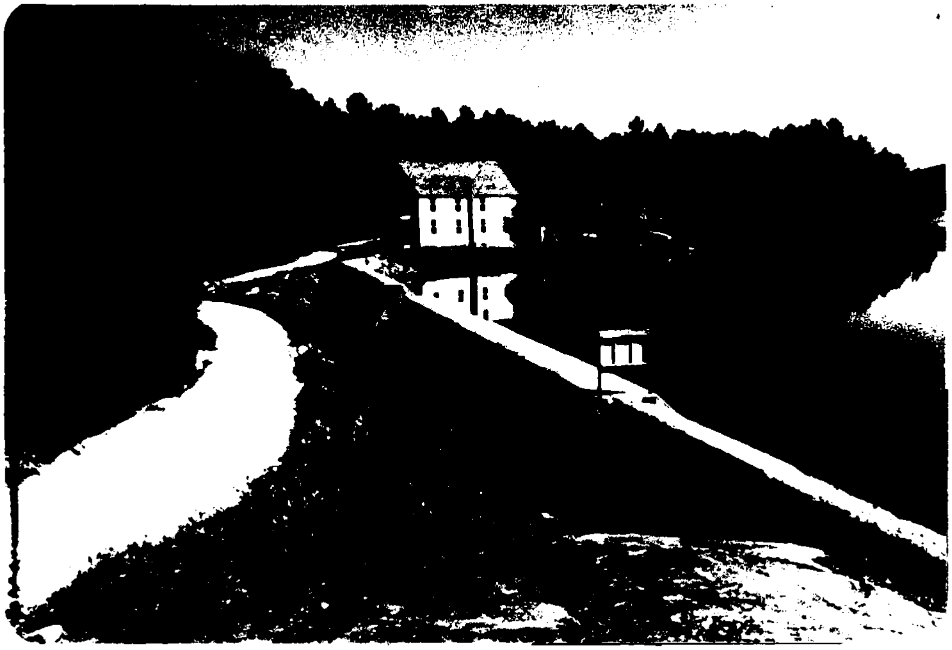


Overview from left abutment showing
bedrock outcrop

Overview from left side downstream
showing spillway

Overview from left side upstream

-SCALE-

0 1/2 1 2 miles

FROM: USGS MASCOMA, N.H.
QUADRANGLE MAP

GOLDBERG, ZOINO, DUNNICLIFF & ASSOC, INC.
GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS.

U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCUS PLAN

GRAFTON POND DAM — NEW HAMPSHIRE

FILE No. 2067

			SCALE	AS NOTED
			DATE	SEPT 1978

PHASE I INSPECTION REPORT

GRAFTON POND DAM

SECTION 1

PROJECT INFORMATION

1.1 General

(a) Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD) has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed was issued to GZD under a letter of August 22, 1978 from Colonel Ralph T. Garver, Corps of Engineers. Contract No. DACW 33-78-C-0303 has been assigned by the Corps of Engineers for this work.

(b) Purpose

(1) Perform technical inspection and evaluation of non-federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-federal interests.

(2) Encourage and prepare the states to initiate quickly effective dam safety programs for non-federal dams.

(3) Update, verify and complete the National Inventory of Dams.

(c) Scope

The program provides for the inspection of non-federal dams in the high hazard potential category based upon location of the dams and those dams in the significant hazard potential category believed to represent an immediate danger based on condition of the dam.

1.2 Description of Project

(a) Location

The Grafton Pond Dam lies on a tributary of the Mascoma River approximately 3.3 miles east of Enfield Center, New Hampshire. The dam is reached via an unnamed dirt road off Route 4A, 2.5 miles south of the village. The portion of the USGS Mascoma, NH quadrangle presented on page ix shows this locus. Figure 1 of Appendix B is a site plan developed from the map and the site visit.

(b) Description of Dam and Appurtenances

The dam at the outlet of Grafton Pond is an Ambursen type concrete structure approximately 285 feet long (Page B-4). It consists of a 40 foot end wall on the left bank, 6 adjacent buttress bays with a total length of 75 feet, a spillway crest 24 feet long which spans over two buttress bays, an additional eight buttress bays with a total length of 100 feet and an end wall on the right bank 45 feet long. The left end wall is located on the axis of the dam, while the right wall is splayed approximately 30° in the direction of the reservoir. Twin sluice gates are located within the right bay below the spillway crest (Pages B-4 and B-6).

The front face of this dam slopes on an incline of 45° and varies in thickness from 12 inches at its top elevation to 18 inches at its base (Pages B-5 through B-7). In general, the buttresses supporting the face slab are 14 inches thick, with the exception of the buttresses at either end of the spillway crest and those adjacent to the end walls which are 16 inches thick. The twin steel sluice gates, which are constructed on an incline parallel to the face of the dam, are 2 feet wide and 3 feet high and are equipped with non-rising brass stems and operated with hand wheels. The gates were manufactured by Caldwell and Wilcox Company, Newbury, New Hampshire.

A bridge structure (Page B-4) is located approximately 12 feet downstream of the spillway crest.

(c) Size Classification

The dam's maximum impoundment of 3940 acre-feet falls within the 1000 to 50,000 acre-feet range which defines INTERMEDIATE size category as defined in the "Recommended Guidelines."

(d) Hazard Potential Classification

The lack of any development downstream of the dam for a distance of over 3 miles leads to a LOW hazard potential classification. The only potential damage would be to small bridges on unimproved roads.

(e) Ownership

The NHWRB owns this dam. Key officials are: Chairman George McGee, Chief Engineer Vernon Knowlton, Assistant Chief Engineer Donald Rapoza and Staff Engineer Gary Kerr. The Board's telephone number is (603) 271-3406 and it can also be reached through the State Capitol operator at (603) 271-1110. The NHWRB assumed ownership in 1970 from the Granite State Electric Company, a division of New England Power Company.

(f) Operator

The NHWRB has a permanent operator who visits the dam weekly. All dam operations are directed by the Board. The operator can be contacted through the individuals listed in subparagraph 1.2(e) above.

(g) Purpose of Dam

The dam's primary purpose is to maintain the level of Grafton Pond for recreational use. Some secondary flood control benefits are also derived.

(h) Design and Construction History

The dam was designed and built in 1918 by the Mascoma River Improvement Company as part of an overall hydroelectric development project. The dam has undergone no significant alterations since its construction.

(i) Normal Operational Procedure

The pond is maintained at or slightly above the crest of the spillway during the summer recreational period. In early fall, the Board draws the pond down approximately 8 feet in anticipation of fall storms and spring runoff. The drawdown remains in effect until February, when the pond is allowed to refill to the recreational level.

1.3 Pertinent Data

(a) Drainage Area

Grafton Pond receives runoff from 3.6 square miles of moderately sloping, heavily forested terrain. There is no development around the shores of the pond.

(b) Discharge at Damsite

(1) Outlet Works

The dam's outlet works consist of two 2 feet wide by 3 feet high steel sluice gates equipped with non-rising brass stems and operated by hand wheels. The inverts of the gates are at El. 1223.5.

(2) Maximum known flood at damsite

Records maintained by the NHWRB indicate that the maximum water level since the advent of state ownership was 1.2 feet below the dam crest in December 1973. With both gates open, this pond level would create a discharge of approximately 420 cfs.

(3) Spillway capacity at maximum pool elevation:

370 cfs at El. 1244.5

(4) Gate capacity at normal pool elevation:

233 cfs at El. 1241.5

(5) Gate capacity at maximum pool elevation:

252 cfs at El. 1244.5

(6) Total discharge capacity at maximum pool elevation:

622 cfs at El. 1244.5

(c) Elevation (feet above MSL)

(1) Top of dam: 1244.5

(2) Maximum pool: 1244.5

(3) Recreational pool: 1241.5

(4) Spillway crest: 1241.5

(5) Streambed at centerline of dam: 1223.5

(6) Maximum tailwater: Unknown

(d) Reservoir

(1) Length of recreational pool: 1.1 miles ±

(2) Storage - recreational pool: 3000 acre-feet ±
- maximum pool: 3940 acre-feet ±

(3) Surface area - recreational pool: 235 acres ±

(e) Dam

(1) Type: Concrete buttress (Ambursen)

(2) Length: 285 feet

(3) Height: 21 feet ±

(4) Top Width: 2.5 feet ±

(5) Side slopes - U/S 1:1
- D/S Vertical

(6) Grout curtain: Unknown

(7) Cutoff: Plans indicate concrete cutoff at upstream toe

(f) Spillway

(1) Type: Concrete ogee

(2) Length of weir: 24 feet

(3) Crest elevation: 1241.5 ±

(4) U/S channel: Broad approach from lake

(5) D/S channel: Narrow, passing under a bridge 12 feet downstream of spillway

(g) Regulating outlets

As mentioned previously, the dam's only regulating outlets are the two steel sluice gates. Subparagraph 1.3(b) presents pertinent data on the gates.

SECTION 2 - ENGINEERING DATA

2.1 Engineering Records

The design of this dam is quite innovative in that it uses a buttress arrangement to eliminate much of the concrete that would normally go into a gravity structure. None of the original design drawings or calculations are available.

2.2 Construction Records

Appendix B contains the construction drawings for the dam. Additionally, the files of the NHWRB contain photographs from 1918 showing the progress of work on the dam. Both sources of information are quite detailed.

2.3 Operational Records

The present owner operates the dam in a manner consistent with its engineering features and intended purpose. Operating records since the advent of state ownership in 1970 are maintained at the Board's Concord office.

2.4 Evaluation

(a) Availability

The available data are quite extensive and therefore warrant a satisfactory evaluation for availability. The lack of design data is not a serious deficiency as the plans are sufficiently detailed to permit backfiguring of engineering data if desired.

(b) Adequacy

The available information is sufficient to permit an evaluation of the dam from the standpoint of reviewing design and construction data. A satisfactory assessment for adequacy is therefore warranted.

(c) Validity

The available construction plans are generally in agreement with the as-built configuration of the dam. Thus, a satisfactory evaluation for validity is assigned.

SECTION 3 - VISUAL OBSERVATIONS

3.1 Findings

(a) General

The Grafton Pond Dam is in VERY POOR condition at the present time. This evaluation is based primarily upon the significant deterioration of the dam's key structural elements, the dam facing wall and its supporting buttresses.

(b) Dam

(1) Left End Wall

Inspection of the left end wall reveals that the top of this wall is severely spalled over 70% of its surface. This spalling can be attributed to moisture intrusion combined with alternating freeze and thaw cycles.

(2) Dam Facing Wall

There is minor surface erosion over the entire facing wall of the structure. This surface erosion can be attributed to ice damage.

The six buttress bays between the left end wall and the spillway and the eight buttress bays between the spillway and the right end wall have expansion joints at alternate bays. These expansion joints consist of steel tee sections embedded in the concrete. The first expansion joint adjacent to the left bank has spalled from the top of the dam to approximately 10 inches below the spillway crest, exposing the steel tee sections. The next expansion joint has similarly spalled from the top of the dam to a point approximately 10 feet lower. The continuation of this joint on the top surface of the dam has opened and spalled. This spalling can also be attributed to moisture intrusion coupled with alternate freeze and thaw cycles. This mode of failure is typical for all expansion joints.

There is spalling at the first expansion joint located adjacent to the right side of the spillway. The concrete has spalled over a distance of approximately 4 feet measured along the sloping face of the structure; the metal expansion tee is exposed and undermined.

The second expansion joint located to the right of the spillway has spalled as deep as the metal tee from the top of the dam to a point some 12 feet down the face of the dam. The third expansion joint has a spall approximately 6 feet long measured from the top of the dam down its face. Again, the steel expansion tee is exposed. The fourth expansion joint has spalled similar to the second.

Surface spalling approximately 4 inches wide by 4 feet long and 1 inch deep has occurred at the top of the facing wall adjacent to the right end wall. This spalling can be attributed to excessive troweling of the concrete and to moisture intrusion coupled with alternate freeze and thaw cycles. There is a horizontal crack on the front face of the wall located in the first buttress bay adjacent to the right abutment. This crack is approximately 1/4 inch wide and extends through the facing wall. The crack is approximately 3 feet below the top of the dam. Approximately 2 feet from the right end wall there is a vertical crack along the facing wall surface.

Investigations of the underside of the facing wall reveal a considerable amount of horizontal cracks, seepage, leaching, stalactites, exudation, and encrustation. In general, the horizontal cracks are located in well defined patterns, suggesting that these cracks occur at the locations of construction joints. The enclosed photographs illustrate the intensity of the facing wall deterioration. Due to the structural hazard condition, no attempt was made to chip away deteriorated concrete for the purpose of investigating the degree of oxidation of the steel reinforcement. However, it can be assumed that oxidation of steel reinforcement does exist.

Construction drawings indicate that the facing wall was designed as a two span section without provision for negative reinforcement over its intermediate support (buttress). Assuming the water surface level is at the spillway crest elevation, preliminary structural calculations reveal that concrete tensile stresses on the exterior of this slab over the intermediate buttress are well beyond its ultimate tensile capacity.

In view of this fact, it can be predicted that vertical cracks have developed in the facing wall (below the water line) over the intermediate supporting buttress. Once cracks develop at these intermediate supports, the facing wall reverts to a simple span structure with open vertical joints as opposed to a two span continuous structure. This situation results in the redistribution of inherent stresses.

Suspected cracking of the facing wall over the intermediate buttresses correlates with observed seepage around the upper surfaces of buttress to slab connections between elevations 1234.5 and 1225.5 (MSL). Furthermore, the construction drawings reveal that a deliberate effort was made to preclude the bonding of the buttresses and facing walls, which suggests a further source for the development of seepage.

Diagonal cracks and spalls are evident at five locations on the back face of the header beams supporting the top of the dam. The first buttress adjacent to the left end wall exhibits diagonal cracks and dislodged concrete. The concrete which has been displaced is triangular in shape and approximately 16 inches on a side. There is additional cracking at the upper end of this buttress and reinforcing steel is exposed. The failures at these locations can be attributed to a lack of restraint against expansion.

(3) Spillway

The spillway crest exhibits a series of longitudinal hairline cracks and minor surface erosion. The hairline cracking can be attributed to overtrowelling the spillway surface in order to obtain a granolithic finish. The minor surface erosion can be attributed to cavitation and to the flow of ice over the spillway. The steel expansion joint adjacent to the left side of the spillway is exposed from its upper terminus downward as far as is visible below the water surface. The erosion at this joint is in excess of 6 inches in width over its exposed length.

The corner of the front facing wall adjacent to the spillway has cracked for a distance of 30 inches above the spillway crest in line with the expansion tee. A block of concrete approximately 18 inches high, 8 inches wide and 12 inches deep has cracked away from the buttress wall adjacent to this particular tee.

(4) Buttresses

Inspection of the buttresses reveals considerable pattern cracking, seepage, erosion, disintegration, scaling, leaching, stalactites, exudation and encrustation. The deteriorated condition of these structural components places the dam in jeopardy. Separate plans locating these deficiencies are included in Appendix B.

The disintegration of concrete at the base of the buttresses is pronounced. In one particular instance, this disintegration is approximately 8 feet in height, 2 feet in width and over 8 inches deep. This deterioration represents a 50% loss in cross section. It is apparent from the visual observations that the exposed surface of the concrete at this particular location is unsound. This condition also exists at the base of the buttress adjacent to the left side of the spillway. Similar types of erosion, but to a lesser degree, are evident in 8 of the remaining 16 buttresses. There is evidence of seepage between all the buttresses and the facing wall. Buttresses Nos. 11, 12, 13 and 14 exhibit only minor seepage, but this continuous flow through the facing wall onto the buttresses over the remaining portion of the structure, though low in terms of volume, is extremely damaging as evidenced by the high degree of erosion. Drain holes were cut through various buttress bases subsequent to construction for the obvious purpose of avoiding ponding adjacent to the buttresses. In general, the erosion of concrete between the buttresses and the facing wall can be attributed to continuous seepage subjected to alternate freeze and thaw cycles. The inventory of the locations of erosion indicates that the highest degree of erosion occurs at the locations of steel expansion joints.

Although erosion at the intermediate buttresses has occurred to a lesser extent, it can be assumed that vertical cracks have occurred in the facing wall over these buttresses below the water line.

The exposed surface of the buttress adjacent to the left side of the spillway has failed structurally. The concrete at this location is badly cracked from 12 inches above to 30 inches above the spillway. The joint between the buttress and the spillway surface has eroded over its entire length. This erosion is approximately 4 inches wide and 2 inches deep. The face of this buttress is also badly checked with a high degree of efflourescence. A triangular concrete section approximately 8 inches on a side and located at the corner of this buttress has been dislodged. There is a conical hole 5 inches deep adjacent to this dislodgement. Expansive forces have also caused the concrete to shear at this location, probably due to lack of reinforcing steel at the connection between the end of the slab and the buttress. Spalls, cracks, checking and efflourescence can be attributed to moisture intrusion coupled with alternate freeze and thaw cycles.

The buttress adjacent to the right side of the spillway exhibits deterioration similar to the left side. This deterioration, which can be attributed to the spalling of concrete over the expansion joint tee, has progressed along the total depth of the concrete facing wall for a distance in excess of 2 feet above the spillway crest. This spalling varies in width from 6 inches at its base to 3 inches at its highest elevation. The connection between the header beam supporting the rear top surface of the facing wall between buttresses exhibits a spall approximately 8 square inches in surface area and 3 inches deep on its face adjacent to the spillway. This spall can be attributed to expansive forces.

(5) <u>Right End Wall</u>

There is a diagonal crack at the top corner of the right end wall immediately adjacent to the first buttress.

This crack, which is approximately 2 feet long, can be attributed to expansive forces. The top of the right end wall exhibits spalling over approximately 10% of its surface area. This spalling can be attributed to moisture intrusion and alternate freeze and thaw cycles. A vertical construction joint located approximately 15 feet from the right buttress has spalled and opened. This spalling can also be attributed to moisture intrusion and alternate freeze and thaw cycles.

(6) Abutments

The left end wall of the dam is in contact with a massive granitic outcrop. The rock appears competent and the widely spaced jointing tightens quickly with depth. There is a small amount of seepage of the junction of the wall with the bedrock outcrop.

The foundation under the right end wall is not visible, but several nearby outcrops suggest that this wall is also founded on competent, granitic material. There is also a small seepage at this abutment.

(7) Foundation Conditions

Observations of the site indicate that the dam is founded on bedrock. This conclusion is consistent with the use of an Ambursen type dam at the site, as this structural system generally requires bedrock support for the buttresses. Photos taken in 1918 and on file with the NHWRB confirm the bedrock foundation and provide an excellent view of the foundation conditions.

(8) Sluice Gates

Inspection of the sluice gates in the presence of a representative of the New Hampshire Water Resources Board indicated that they were extremely difficult to operate. A two-man team attempted to turn the wheels, but was unsuccessful. The inspection team was advised that a 2 by 4 wood fulcrum still required two men and considerable effort to turn this wheel two revolutions, which would raise the gates 1/8 inch.

The bench stands for these sluice gates are located on a plane 45° to that of the floor and parallel to the face of the dam. The left pedestal base exhibits seepage and surface erosion over an area 18 inches square. The right pedestal base also exhibits a minor degree of seepage. The surface erosion can be attributed to moisture intrusion and alternate freeze and thaw cycles.

(9) Downstream Bridge

There is erosion and undermining of the wall extension of the right abutment of the downstream bridge. The eroded area is 6 feet long, 3 feet wide and up to 2 feet deep. This erosion can be attributed to cavitation of the concrete. The left abutment shows no evidence of undermining or erosion.

(c) Appurtenant Structures

The dam has no appurtenant structures.

(d) Reservoir

Examination of the reservoir shore revealed no evidence of instability or potential slides. No sedimentation was noted behind the dam and there was no evidence of work in progress or recently completed which might increase the flow of sediment into the pond. There were no indications of any changes to the surrounding watershed which might adversely affect the runoff characteristics of the basin. There is only one home, located near the right abutment of the dam, within approximately one half mile of the pond.

(e) Downstream Channel

Discharge from the dam passes under a bridge 12 feet downstream and then into a well defined, heavily overgrown channel. There is no development along the channel for at least 3 miles downstream. Some local roads could, however, be affected by a dam failure. There are no downstream conditions which would limit operation of the dam.

3.2 Evaluation

The dam is in VERY POOR condition at the present time and requires immediate corrective action. Fortunately, however, the dam poses only the most remote hazard to downstream life and property.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures

As mentioned previously, a NHWRB operator visits the dam at least weekly and reports gage readings back to the Concord office. Engineers at the office, in turn, direct any gate operations necessary. The lake is maintained at or slightly above the spillway crest during the summer and is drawn down 8 feet in early fall in anticipation of fall storms and spring runoff.

4.2 Maintenance of Dam

The dam operator inspects the dam during his weekly visits and reports any deficiencies back to the Board's engineers, who take the necessary steps to institute repairs. Additionally, an engineer from the Board inspects the dam annually.

4.3 Maintenance of Operating Facilities

The same procedures outlined above apply to the dam's operating facilities. The gate stems were replaced in the 1971 to 1972 time frame.

4.4 Description of any Warning System in Effect

No formal warning system exists for this dam.

4.5 Evaluation

The present condition of the dam indicates a long period of neglected maintenance. It may, in fact, be too late to arrest further deterioration. As there is no appreciable downstream hazard from a dam failure, the lack of a formal warning system is not a significant problem.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

(a) Available Data

Data sources available for Grafton Pond Dam are "Data on Dams in New Hampshire" and "Data on Reservoirs and Ponds in New Hampshire," both by the New Hampshire Water Control Commission and dated December 6, 1938. These sources contain basic information concerning the dam structure and dimensions. Some of the original hydraulic calculations, which include the maximum waste gate and spillway capacities prior to overtopping of the dam, are also available.

(b) Experience Data

Flood experience data for the dam indicate a maximum water level in the pond of 1.2 feet below the dam crest during a storm in December 1973.

(c) Visual Observations

Grafton Pond Dam is a buttressed concrete structure known as an Ambursen Dam after its designer. The pond and dam are located in Grafton, New Hampshire and discharge into a tributary of Bicknell Brook, which flows towards the village of Lockehaven. The dam has an overall crest length of 285 feet at a height of 3 feet above its spillway. The dam has a maximum height of about 21 feet above the stream bed with a top width of about 2.5 feet.

The discharge works consist of a 24 foot long broad-crested concrete spillway and two 2 foot by 3 foot sluice gates. The gates are operated from a gate house built under the spillway. The elevation of the spillway crest is given in the available data as 1241.5 feet above Mean Sea Level (MSL). The invert elevations of the sluice gates on this basis are therefore at 1223.5 feet. At the time of the inspection the water level in the pond was about four feet below the spillway crest, or at about elevation 1236.5 feet.

Immediately downstream of the dam is a roadway embankment with an 11.0 foot by 11.5 foot bridge opening for the stream. The top of the embankment and bridge are about 4.5 feet lower than the spillway, or at elevation 1237.0.

Although this opening is large enough to pass peak gate and spillway flows, it would very likely be washed away in the event of a dam failure. Beyond this point, the only other structures in the flood hazard area are two other small bridges. No homes or other buildings are located in the region that would be inundated.

(d) Overtopping Potential

The hydrologic conditions of interest in this Phase I investigation are those required to assess the dam's overtopping potential and its ability to safely allow a large flood to pass. This analysis requires use of the storage and discharge characteristics of the structure to evaluate the impact of an appropriately sized Test Flood. The available discharge capacity calculations were used as a check on the calculations developed for this purpose.

Guidelines for establishing a recommended Test Flood based on the size and hazard potential classifications of a dam are specified in the "Recommended Guidelines" of the Corps of Engineers (COE). As shown in these guidelines, the appropriate Test Flood for a dam classified as INTERMEDIATE in size with a LOW hazard potential would be between the 100 year frequency flood and one-half of the Probable Maximum Flood (PMF).

The magnitude of the 100 year peak inflow to Grafton Pond is estimated using a regression relationship provided by the USGS in Water Resources Investigations 78-47, "Progress Report on Hydrologic Investigations of Small Drainage Areas in New Hampshire." This equation, which uses the drainage area, main channel slope and the 24-hour, 2-year frequency precipitation to estimate peak inflow, yields a 100 year peak flood flow of 780 cfs for the Grafton Pond drainage basin.

The chart of "Maximum Probable Peak Flow Rates" obtained from the Corps of Engineers, New England Division is used to determine the PMF. For the 3.6 square mile drainage area above Grafton Pond, which has a hilly topography, the curve for "rolling" terrain gives a PMF flow of 1925 cfs per square mile. This value results in a total PMF of 6,930 cfs, or a one-half PMF flow of 3,470 cfs.

The "Guidelines" further suggest that if a range of values is indicated for the Test Flood, the magnitude most closely related to the involved risk should be selected. Since the risk is towards the lower end of the LOW category, a Test Flood of 1800 cfs is used as inflow to Grafton Pond.

The attenuation of the peak due to storage is estimated using the procedure suggested by the Corps of Engineers, New England Division for "Estimating the Effect of Surcharge Storage on Maximum Probable Discharges." The Storage-Stage Curve used for these calculations is developed assuming that the surcharge storage available in a pond is equal to the surface area of the pond times the depth of surcharge. No spreading or increase in surface area with increasing depth is considered. Use of the recommended procedure shows that the pond storage has a very significant attenuating effect on the magnitude of the peak flow, since the calculations result in a corrected Test Flood flow of about 900 cfs, or a fifty percent reduction in the pond inflow.

The Stage-Discharge Curve is developed by defining discharge as the sum of the flows through the two gates, flow over the spillway, flow over the dam crest, and the flow over the slopes at the ends of the dam. These calculations assume both gates are fully open. Application of the attenuated Test Flood peak discharge of 900 cfs to the derived Stage-Discharge relationship results in a maximum stage at the dam of about 3.4 feet above the spillway, or about 0.4 feet above the dam crest.

5.2 Hydrologic/Hydraulic Evaluation

The results of the hydrologic and hydraulic calculations indicate that the outlet capacity of Grafton Pond Dam is insufficient to pass a Test Flood flow in the lower range of that suggested by the "Guidelines." The maximum capacity of the waste gates and spillway, with the water level at the dam crest, was computed to be 620 cfs. This quantity is in close agreement with the available previous calculation of 684 cfs.

5.3 Downstream Dam Failure Hazard Estimates

The flood hazards in downstream areas resulting from a failure of Grafton Pond Dam are estimated using the procedure suggested in the COE New England Division's "Rule of Thumb Guidelines for Estimating Downstream Dam Failure Hydrographs." This procedure accounts for attenuation of dam failure hydrographs in computing flows and flooding depths for downstream reaches.

For these calculations, failure is assumed to occur as soon as the dam crest is overtopped at an elevation of 1244.5 feet. This level corresponds to a height of 21 feet above the stream bed. For an assumed breach width of 100 feet, the resultant peak discharge due to dam failure is 16,200 cfs.

Downstream of the dam, there are only three structures within the floodway before the confluence with Bicknell Brook. All three are small bridges which are assumed to fail when struck by the dam failure discharge and therefore would not represent significant impediments to the flood wave. No structures are located in the flood hazard area of Bicknell Brook between this confluence and its point of discharge into Mascoma Lake.

Between Grafton Pond and Bicknell Brook, the floodway is divided into two reaches. The first reach is about 3000 feet long with a well defined channel and fairly steep slope. The second reach is 3700 feet long with a somewhat wider channel and shallower slope.

The first reach attenuates the peak from 16,200 cfs to 15,800 cfs. The depth of flow in this reach is 12.3 feet. The second reach further attenuates the flood flow peak to 15,100 cfs and has a depth of flow of 9.8 feet. These flows and depths of flooding would, in all probability, wash out the downstream bridges, but there are no known residences that would be affected. Although the possibility of some bridge and roadway damage is high, the potential for loss of life is considered remote.

SECTION 6 - STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

(a) Visual Observations

The field investigations of the dam revealed no significant displacements, but considerable structural distress which could influence the stability of the structure is evident.

(b) Design and Construction Data

At least one design feature of the dam may have contributed to its present condition and potential instability. The upstream facing slab was designed as a two span structure and contains waterstops at the construction joints on every other buttress. The slab is continuous over intermediate buttresses, but contains, however, no negative reinforcement at these locations. Preliminary calculations within the scope of a Phase I investigation indicate that the slab is stressed beyond its tensile capacity at these intermediate buttresses. Therefore, vertical cracks likely do exist over the buttresses. These cracks, in turn, are not waterproof and have apparently permitted water to penetrate the slab and to induce deterioration of the slab and, in some cases, the supporting buttresses. Additionally, inherent stresses have been redistributed by the slab changing from a two span continuous structure to a single span, simply supported configuration. Section 3 discusses this situation in greater detail.

The construction drawings included in Appendix B are quite detailed and would facilitate the preparation of a stability analysis were one deemed necessary. The operating records maintained since 1970 reveal no evidence of instability under experienced pond levels.

(d) Post-Construction Changes

There have been no significant post-construction changes to the dam.

(e) Seismic Stability

The dam is located in Seismic Zone No. 2 and, in accordance with recommended Phase I guidelines, does not warrant seismic analyses.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

(a) Condition

The Grafton Pond Dam is in VERY POOR condition at the present time.

(b) Adequacy of Information

Available information is quite extensive and does permit an assessment from the point of view of reviewing design and construction data. This review is a valuable supplement to the visual inspection.

(c) Urgency

The recommendations and remedial measures stated in Paragraphs 7.2 and 7.3 should be initiated within one year of receipt of this report except that the lowering of the pool shall start immediately upon receipt of this report.

(d) Need for Additional Investigation

Additional investigations are required as recommended in Paragraph 7.2.

7.2 Recommendations

It is recommended that a qualified structural engineer be retained to inspect and to analyze the condition of this dam. It is anticipated that such an investigation could result in a recommendation that major repairs be undertaken or that the dam be rebuilt.

7.3 Remedial Measures

Based upon preliminary calculations, it is recommended that the level of Grafton Pond be maintained at a level no higher than gage elevation 9.0 until such time as the reservoir can be completely drawn down to permit the investigation recommended above.

The development of remedial structural measures for this dam, if any exist, is beyond the scope of a Phase I investigation. These measures will more properly result from the investigation recommended above.

The only non-structural remedial measure relates to the dam's operating policy. To decrease response time in the event of an emergency when NHWRB personnel might become over-extended, the Board should train local municipal officials in the proper operation of the dam and establish a procedure for utilizing this manpower resource in the event of unforseen circumstances.

Technical inspections of the dam should continue to be made every year.

APPENDIX A

VISUAL INSPECTION CHECKLIST

INSPECTION TEAM ORGANIZATION

Date: September 20, 1978

NH 00119
GRAFTON POND DAM
Grafton, New Hampshire
Tributary of Bicknell Brook
NHWRB 96.01

Weather: Sunny and warm

INSPECTION TEAM

Robert Minutoli	Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD)	Team Captain
William Zoino	GZD	Soils
Nicholas Campagna	GZD	Soils
Andrew Christo	Andrew Christo Engineers (ACE)	Structural
Paul Razgha	ACE	Structural
Richard Laramie	Resource Analysis, Inc.	Hydrology

Mr. Lyle Milligan of the NHWRB accompanied the inspection team and operated the gates.

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
SUPERSTRUCTURE		
a. General		
Vertical alignment and movement	E.M.	No deficiencies noted
Horizontal alignment and movement		No deficiencies noted
Condition at abutments		Base of left abutment in contact with massive granitic outcrop: rock appears to be competent and widely spaced jointing tightens up quickly with depth: bedrock outcrop also evident at right abutment; rock in this area is also granitic with some pegmatite dikes up to a few inches thick
Rock slope protection		Considerable amount of large boulders (up to 2' ø) has been dumped in front of right endwall near abutment; purpose of rock not evident, but presumably for erosion protection; smaller amount at base of left endwall
Unusual downstream seepage at abutments		Small amount of seepage at both abutments
Foundation conditions		Examination of 1918 photos in the records of the NHWRB confirm that the entire structure is founded on bedrock as shown in the plans

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
Foundation drainage features		None shown on plans or evident at dam
b. Left End Wall	PL	
General condition of concrete		Fair
Rusting or staining		None noted
Spalling		Spalling over 70% of its top surface area
Erosion or cavitation		None noted
Visible reinforcing		None noted
Seepage or efflourescence		None noted
Cracking		None noted
Junction with dam facing wall		Minor spalling
c. Dam Facing Wall - Impoundment Face		
General condition of concrete		Poor
Rusting or staining		At expansion joint locations
Spalling	PL	Concrete cover above steel expansion joints has spalled and undermined these joints; this condition is evident at all expansion joints; extreme right buttress spalled adjacent to the right end wall

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
Erosion or cavitation	TL	Minor surface erosion
Visible reinforcing		Refer to "Buttress"
Seepage or efflourescence		Refer to "Underside of Facing Wall"
Cracking		Horizontal crack full length of right bay; vertical crack 2 feet from right end wall; for additional cracking refer to "Underside of Facing Wall"
d. Buttress Faces Normal to Spillway Axis		
General condition of concrete		Poor
Rusting or staining		At expansion joint locations
Spalling		Isolated structural failures with related spalls, concrete dislodged
Erosion or cavitation		Joints between buttresses and spillway eroded over entire length
Visible reinforcing		None noted
Seepage		None noted
Cracking	TL	Extensive cracking, checking and minor efflourescence

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
e. Buttresses and Underside of Facing Wall	TL	
1. Buttresses		
General condition of concrete		Poor - High degree of erosion at connection with facing walls over a major portion of the dam
Rusting or staining		Considerable staining due to seepage
Spalling		Severe
Erosion		Severe
Visible reinforcing		Connection at top of extreme end of left buttress at header beam
Seepage or efflourescence		Severe - exudation and encrustation
Scaling		Severe
Cracking		Severe random cracking
2. Underside of Facing Wall		
General condition of concrete		Poor - High degree of pattern cracking, seepage, efflourescence, exudation, encrustation and stalactites
Rusting or staining		Considerable staining due to seepage
Spalling	TL	At pattern cracks

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
Erosion		At pattern cracks
Visible reinforcing		None noted
Seepage		2 gpm through longitudinal crack between buttresses Nos. 4 and 5. Seepage in all other buttress bays
Efflourescence		Severe with associated exudation, encrustation and stalactites in all bays
Cracking		Severe pattern cracking in all bays
f. Right End Wall		
General condition of concrete		Fair
Rusting or staining		None noted
Spalling		Vertical construction joint spalled, 10% of top surface area spalled
Erosion or cavitation		None noted
Visible reinforcing		None noted
Seepage or efflourescence		None noted
Cracking		Diagonal crack at top corner adjacent to right buttress; vertical construction joint opened

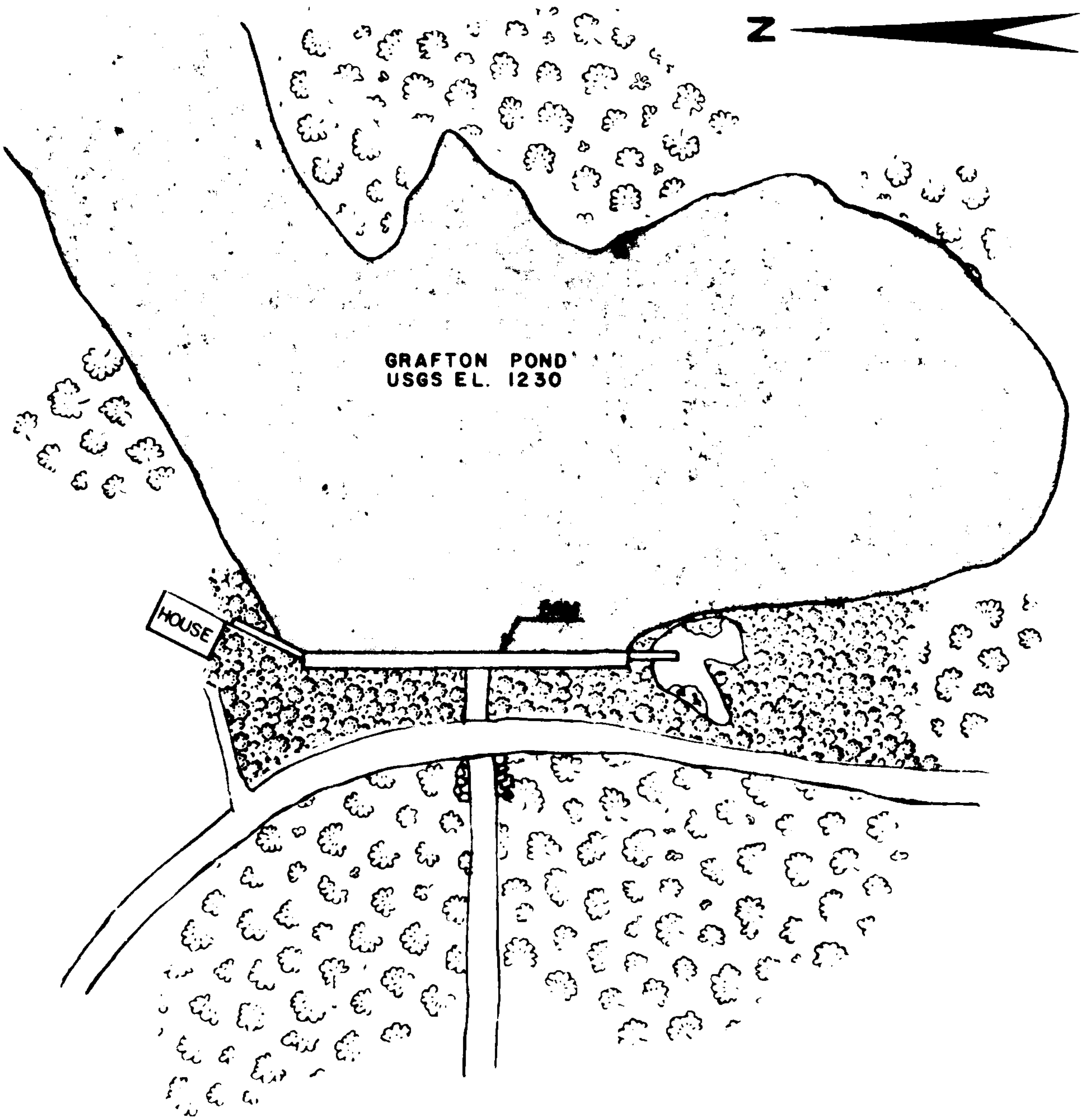
CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
OUTLET WORKS		
a. Approach Channel		
Bottom conditions		Wide, deep approach from pond; bottom not visible
Rock slides or falls		No rock near approach channel
Log boom		None
Control of debris		No debris evident behind dam
Trees overhanging channel		None
b. Spillway		
General condition of concrete		Fair
Rusting or staining		None noted
Spalling		None noted
Erosion or cavitation		Minor surface erosion
Visible reinforcing		None noted
Cracking		Longitudinal hairline cracks
c. Sluice Gates		The sluice gates are extremely difficult to operate; condition unknown; seepage and surface erosion at base of pedestals; gate house locked
d. Downstream Bridge Structure		
General condition		Fair
Right abutment		Wall extension severely eroded

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
Left abutment	PC	Good condition
e. Existence of gage	PC	On upstream face near spillway; spillway crest at 16.0 on gage
OUTLET CHANNEL (immediate area)		
Slope conditions	AL	Discharge flows directly under bridge and into a well defined channel moderately steep slopes
Rock slides or falls		None noted
Control of debris		Some debris downstream which may have washed over dam
Trees overhanging channel		None between dam and bridge
Other obstructions		None noted
RESERVOIR		
a. Shoreline		
Evidence of slides		None noted
Potential for slides		Shoreline stable
b. Sedimentation		None noted
c. Upstream hazard areas in the event of back-flooding		No development along shore of pond
d. Changes in nature of watershed (agriculture, logging, construction, etc.)	AL	None noted

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
DOWNSTREAM CHANNEL		
Restraints on dam operation		None noted
Potential flooded area		Only small road bridges on unimproved roads subject to flooding
OPERATION & MAINTENANCE FEATURES		
a. Reservoir regulation plan		
Normal procedures		Maintain water at spillway level during summer recreational period; draw down 8 feet in late summer or early fall for flood control
Emergency procedures		Permanent dam tender could open gates fully, but response time might be slow due to large number of dams to be covered
Compliance with designated plan		Satisfactory
b. Maintenance		Present condition of dam points to inadequate maintenance effort

APPENDIX B

Note: Pages B-3 through B-5 are Mascoma River Improvement Company drawings dated 1916 modified by GZD to reflect current deficiencies



GOLDBERG, ZOINO, DUNNICLIFF & ASSOC., INC.
GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS.

U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

FIG. 1

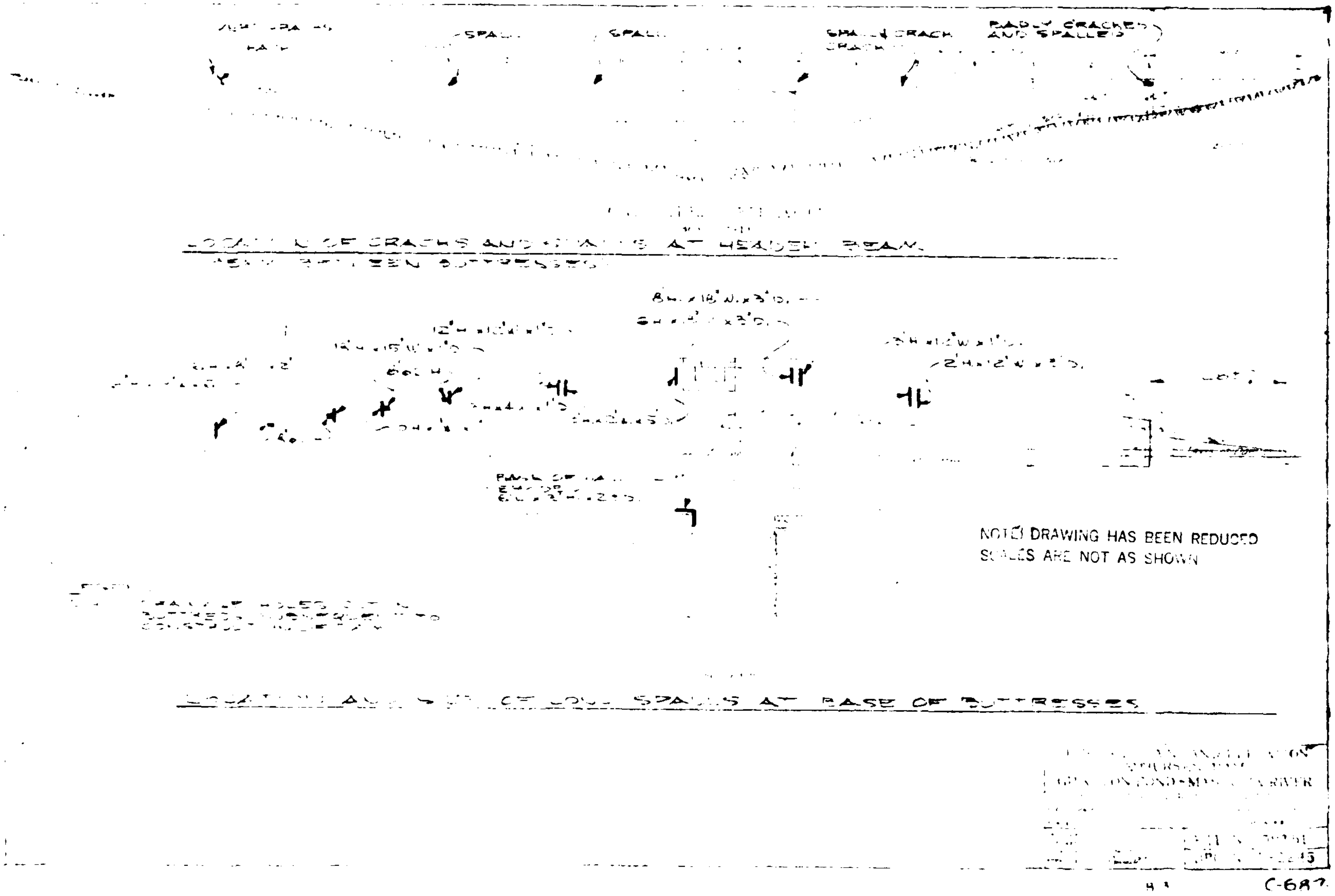
SITE PLAN

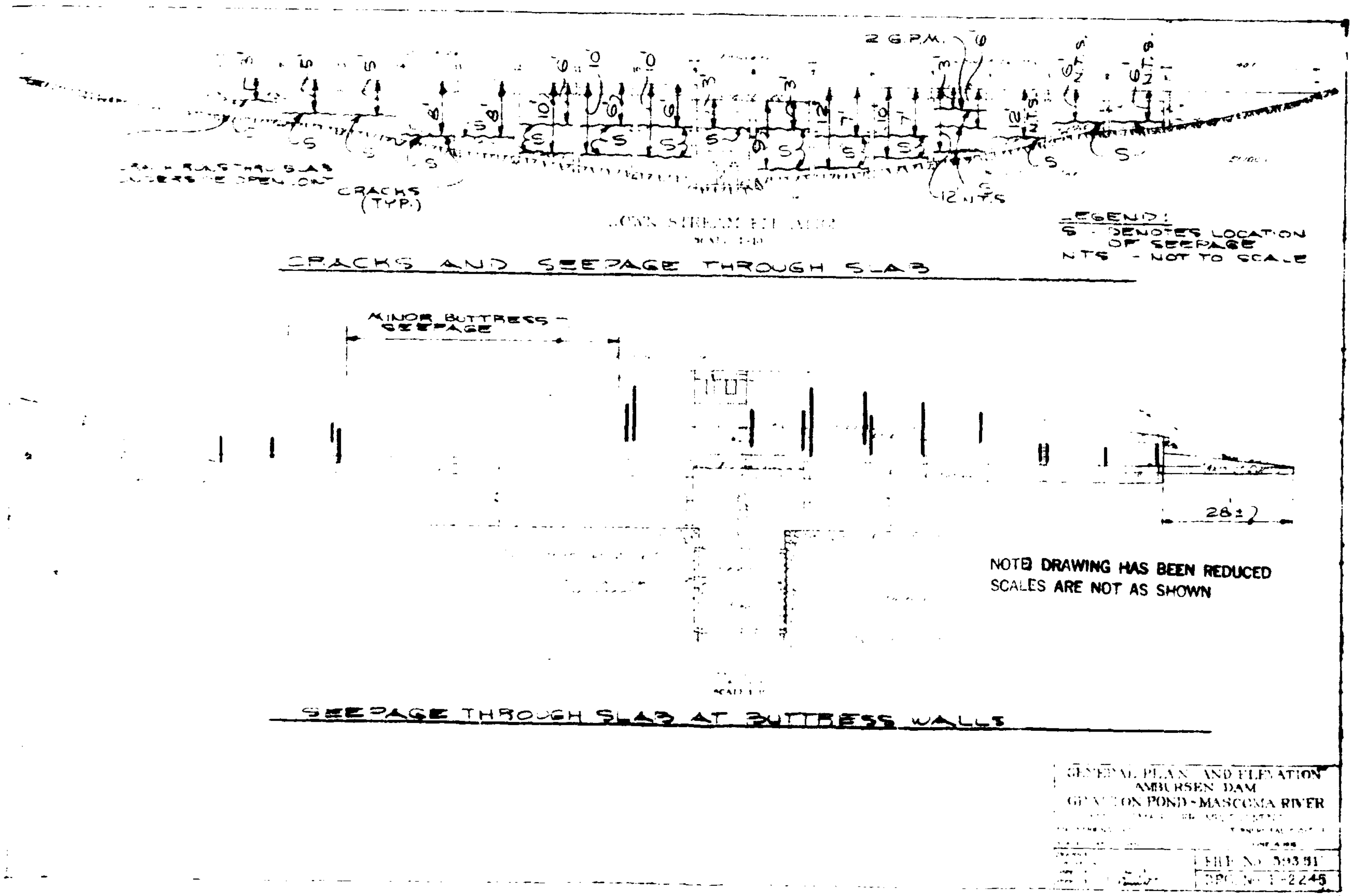
GRAFTON POND DAM — NEW HAMPSHIRE

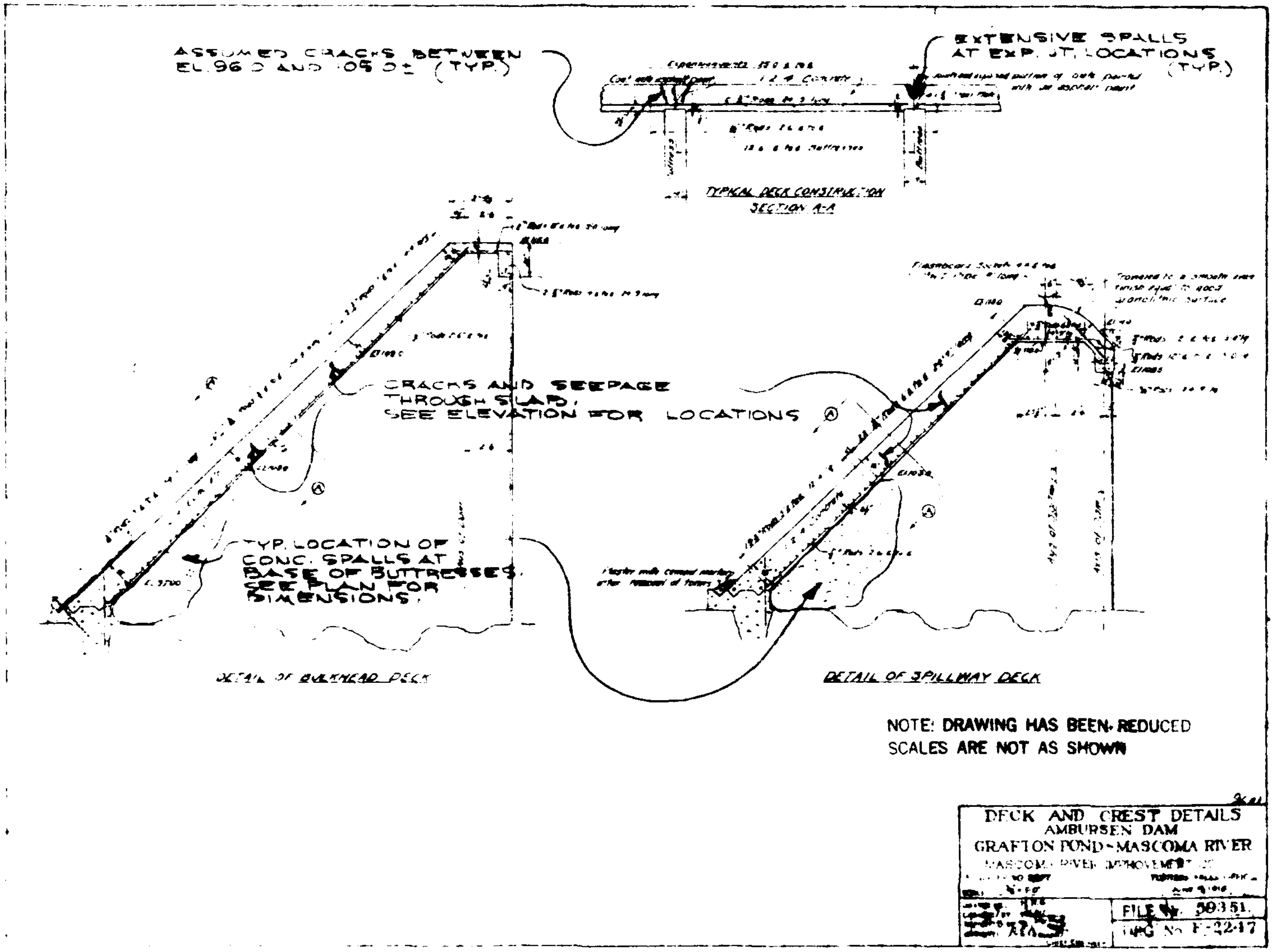
FILE No. 2067

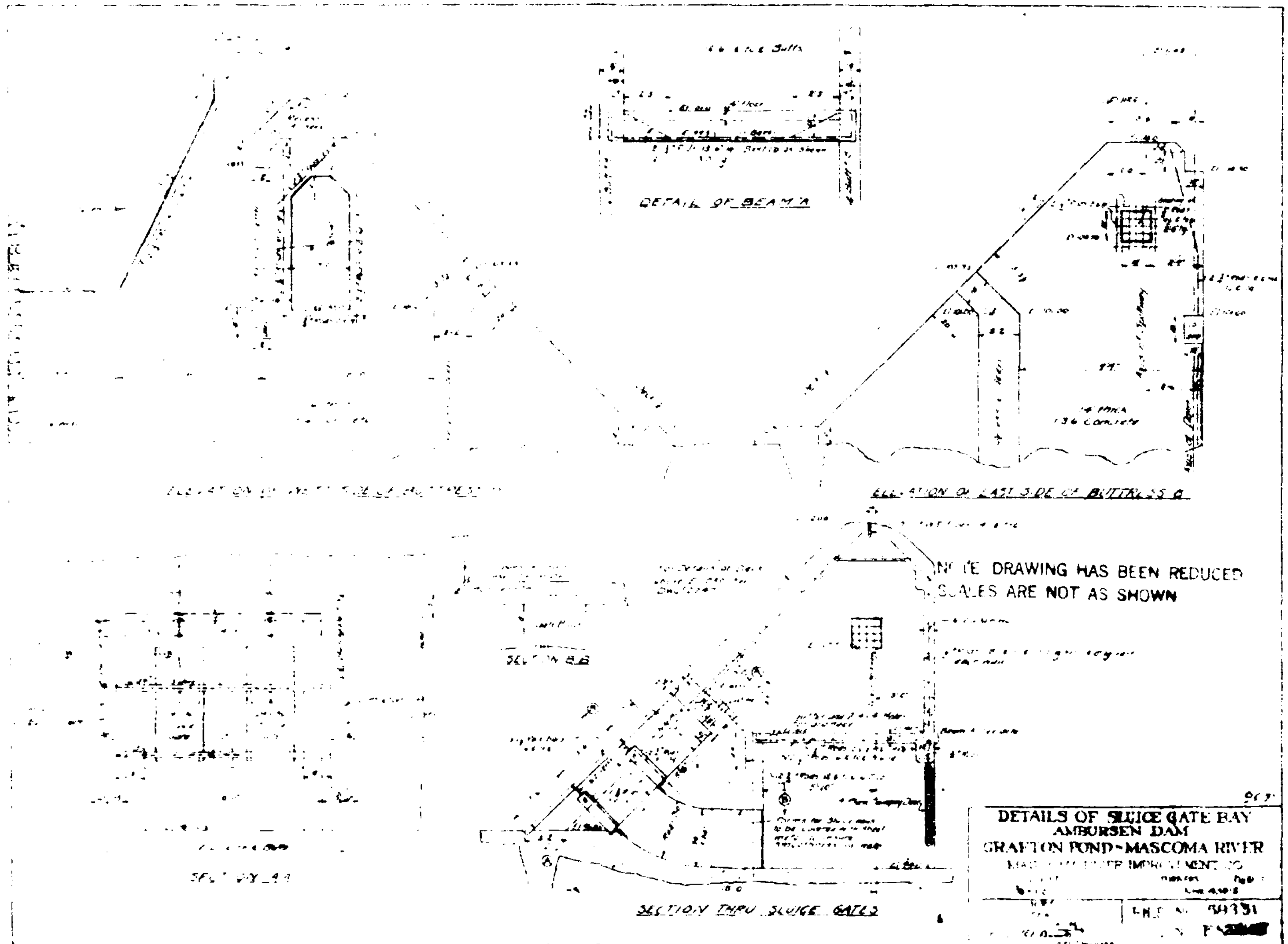
SCALE 1" = 100'

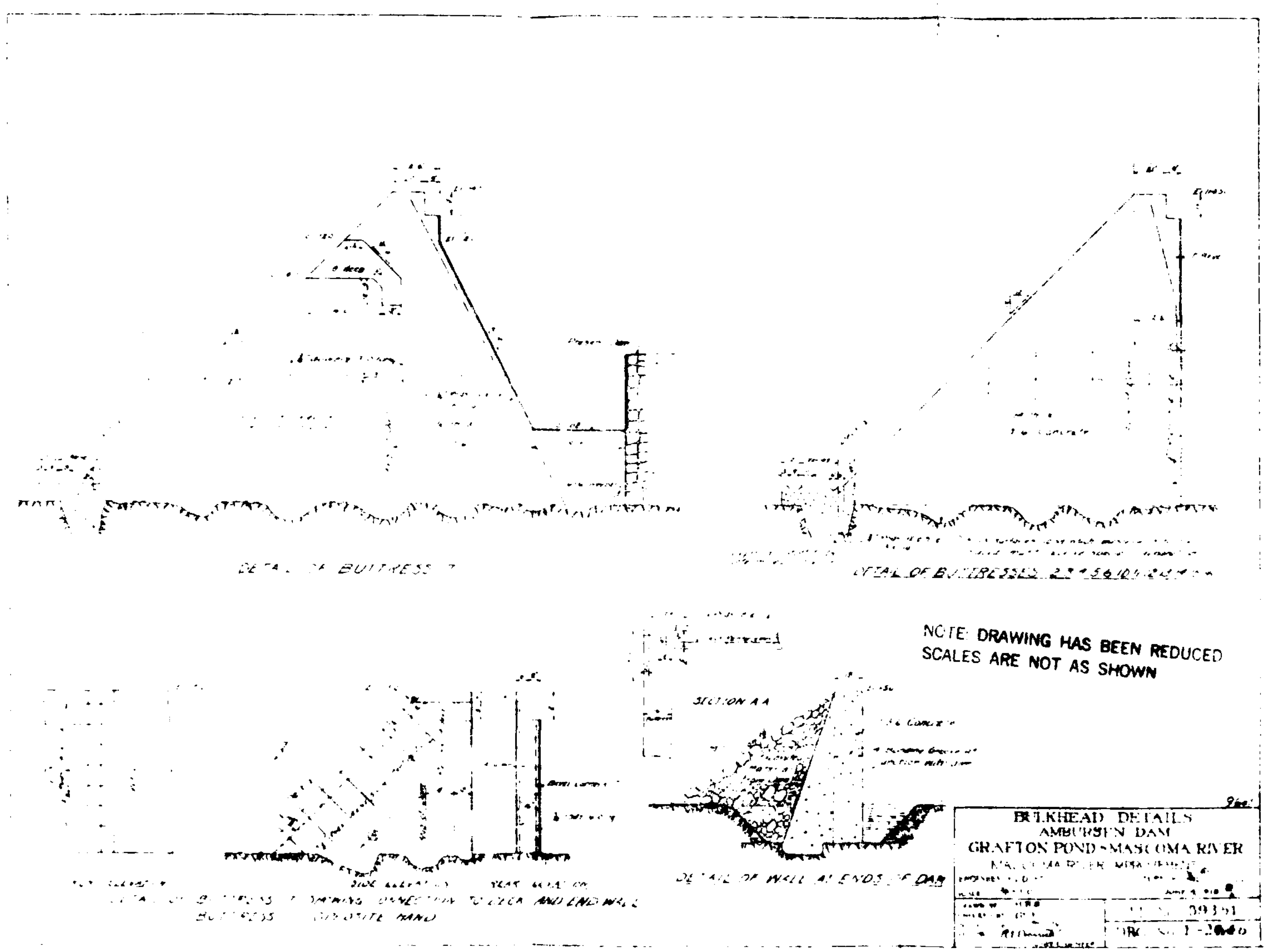
DATE SEPT 1978


BADLY CRACKED AND SPALLED
SPALL
SPALL
SPALL & CRACK
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
C-687


2 G.P.M.
CRACKS (TYP.)
LEGEND:
S - DENOTES LOCATION OF SEEPAGE
NTS - NOT TO SCALE
CRACKS AND SEEPAGE THROUGH SLAB
MINOR BUTTRESS - SEEPAGE
28±
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
SEEPAGE THROUGH SLAB AT BUTTRESS WALLS
GENERAL PLAN AND ELEVATION
AMBURSEN DAM
MASCOMA RIVER
FILE No. 593.91
Q.P.C. No. 1-2245


ASSUMED CRACKS BETWEEN
EL. 96.0 AND 105.0± (TYP.)
EXTENSIVE SPALLS
AT EXP. JT. LOCATIONS
(TYP.)
TYPICAL DECK CONSTRUCTION
SECTION A-A
CRACKS AND SEEPAGE
THROUGH SLAB,
SEE ELEVATION FOR LOCATIONS
TYP. LOCATION OF
CONC. SPALLS AT
BASE OF BUTTRESSES
SEE PLAN FOR
DIMENSIONS
DETAIL OF BULKHEAD DECK
DETAIL OF SPILLWAY DECK
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
DECK AND CREST DETAILS
AMBURSEN DAM
GRAFTON POND - MASCOMA RIVER
FILE No. 59351
DWG No. F-2247


DETAIL OF BEAM A
ELEVATION OF EAST SIDE OF BUTTRESS 8
NOTE DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
SECTION B-B
SECTION A-A
SECTION THRU SLUICE GATES
DETAILS OF SLUICE GATE BAY
AMBURSEN DAM
GRAFTON POND - MASCOMA RIVER


DETAIL OF BUTTRESS 7
DETAIL OF BUTTRESSES
SECTION A-A
DETAIL OF WALL AT ENDS OF DAM
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
BULKHEAD DETAILS
AMBURSEN DAM
GRAFTON POND - MASCOMA RIVER
C 686

The New Hampshire Water Resources Board, 37 Pleasant Street, Concord, NH 03301 maintains a comprehensive correspondence file on the dam dating back to the 1930's. Included in this file are:

(a) Several pages of calculations done by the NHWRB in 1978 for a possible resurfacing of the dam.

(b) One page of calculations done by the NHWRB in 1939 concerning the discharge capacity of the dam.

(c) A 1939 report by the New Hampshire Water Control Commission entitled "Data on Reservoirs and Ponds in New Hampshire."

(d) A 1939 report by the same agency entitled "Data on Dams in New Hampshire."

(e) Operational records since the advent of state ownership in 1970.

(f) Photos from 1918 showing the construction of the dam at various phases.

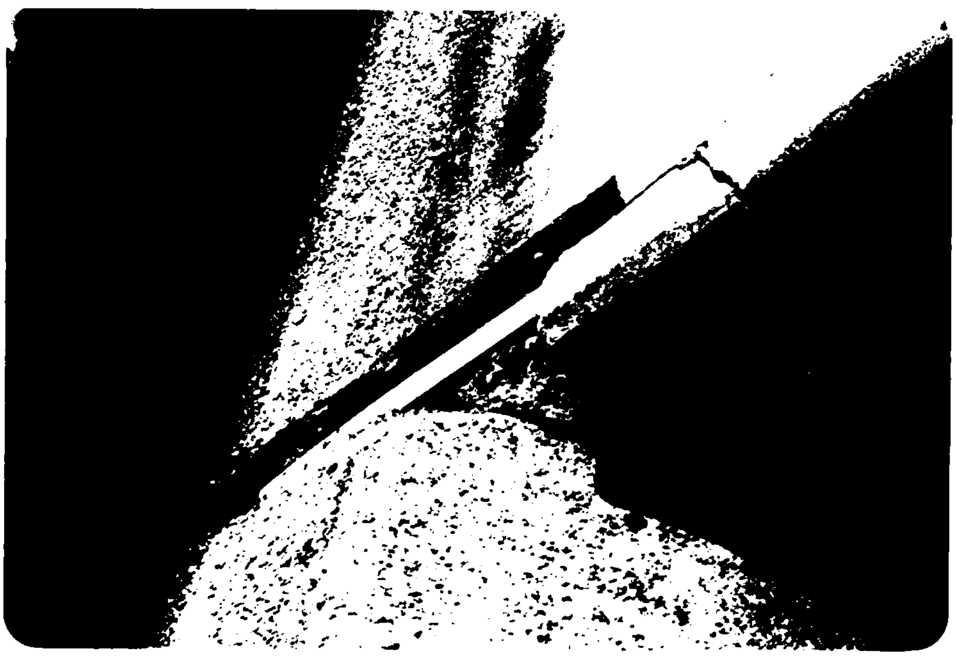
APPENDIX C

SELECTED PHOTOGRAPHS

1. View from downstream of deteriorated concrete at junction of buttress and face of dam

2. View from downstream of deteriorated concrete and efflourescence at junction of buttress and face of dam

3. View from downstream of seepage through dam at junction of buttress and face of dam

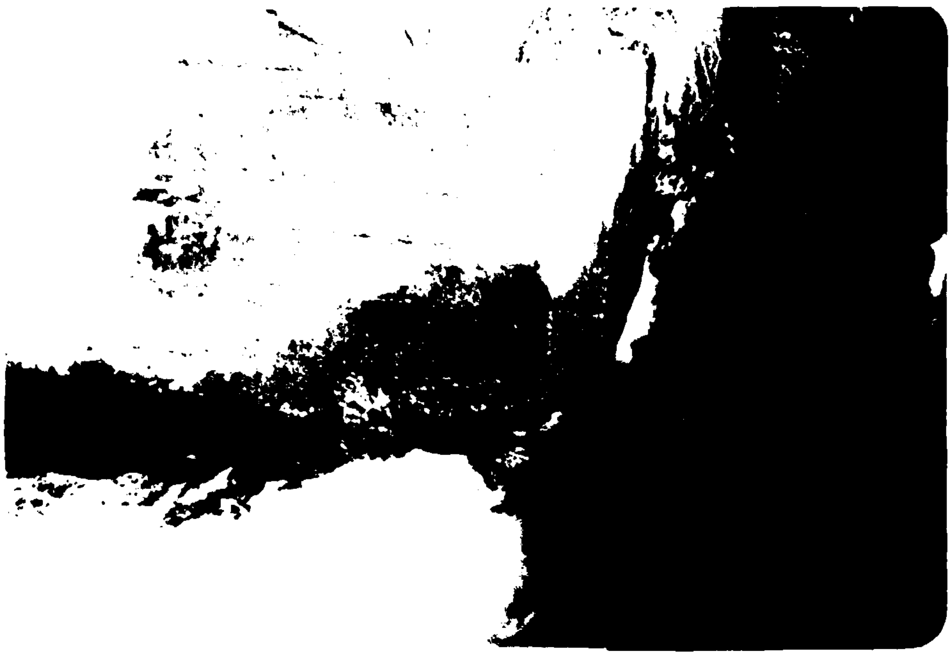


4. View from top of dam showing deteriorated condition of construction joints in face of dam

5. View from downstream showing deterioration and structural failure of concrete at junction of top of dam and buttress

6. View from spillway showing cracking, spalling and efflourescing of concrete

7. View from spillway showing severe erosion of concrete wall immediately downstream of outlet



8. Detail of Photo 7 showing water flowing through corner of downstream wall and road bridge abutment

APPENDIX D

HYDROLOGIC/HYDRAULIC COMPUTATIONS

Grafton Pond Dam

Size classification (f(storage)) = Intermediate
Hazard classification = low

Therefore the appropriate Test Flood is between the 100-yr freq. flood and one-half the PMF

To determine the 100-yr frequency flood the following regression equation is used (LeBlanc, USGS Water Resources Investigations 78-47):

$$P_{100} = 0.55A^{1.05}S^{0.56}I^{2.72}$$

where A = drainage area (mi^2)
S = main channel slope (ft/mi)
I = max. 24-hr precip 80 yr recur. (in.)

A = 3.6 sq. mi.
S = 220 ft/.7 mi = 310 ft/mi
I = 2.7 in

$$P_{100} = 0.55(3.6)^{1.05}(310)^{0.56}(2.7)^{2.72}$$

$$P_{100} = 782 \text{ cfs}$$

The PMF for this basin, with rolling topography and a drainage area of 3.6 mi^2 is

$$PMF = (1925 \text{ cfs}/mi^2)(3.6\ mi^2) = 6930 \text{ cfs}$$

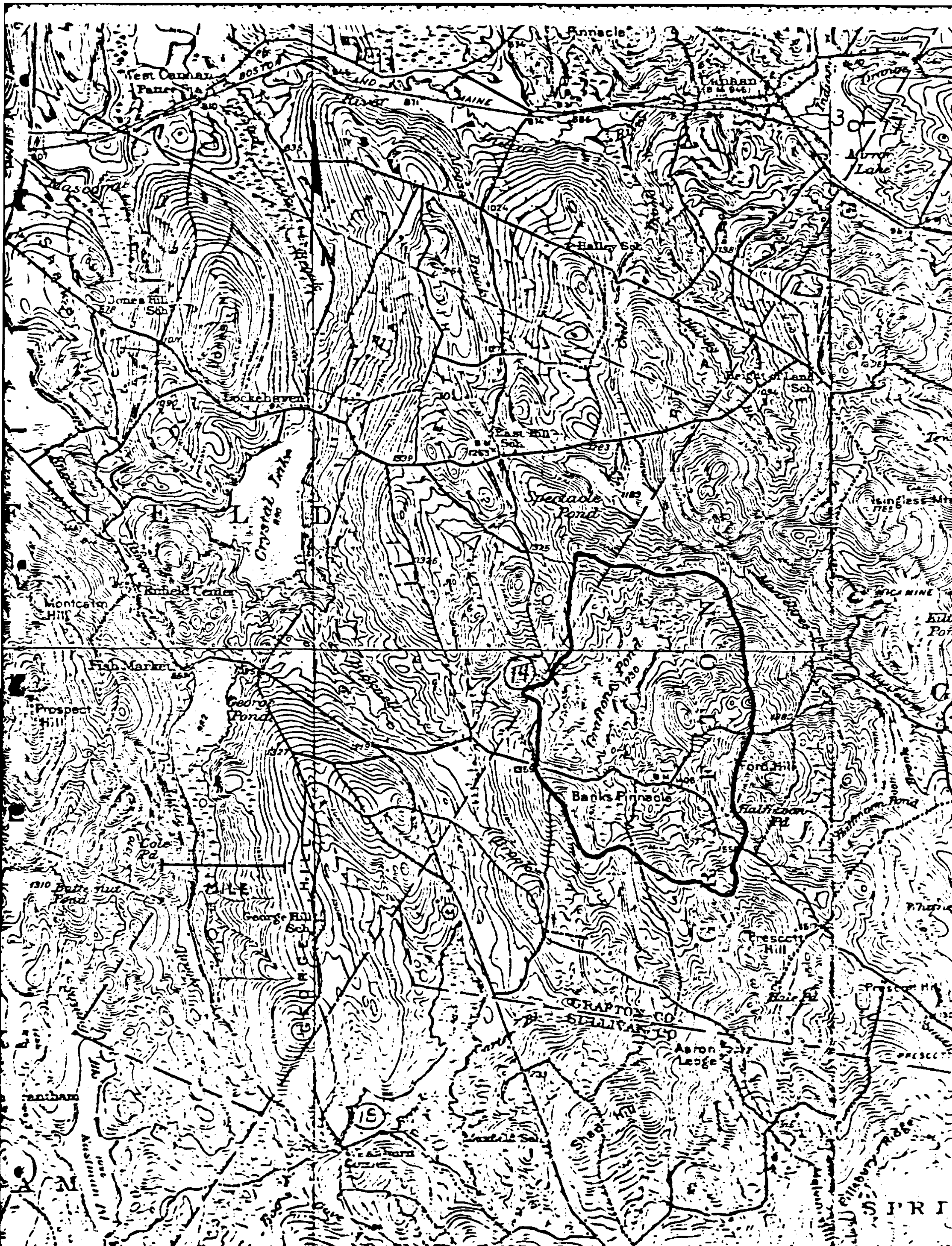
Grafton Pond Dam

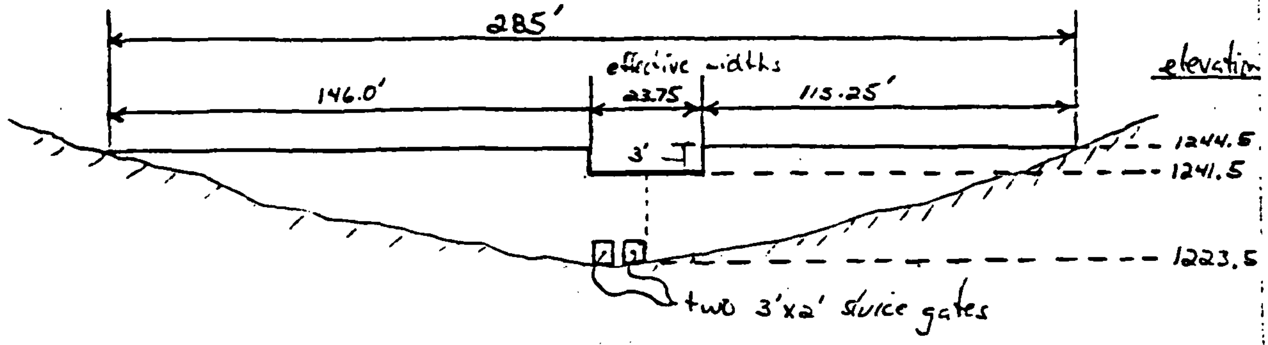
If the PMF is 6930 cfs then ½ the PMF is 3465 cfs which means:

$$782 \text{ cfs} < \text{Test Flood} < 3465 \text{ cfs}$$

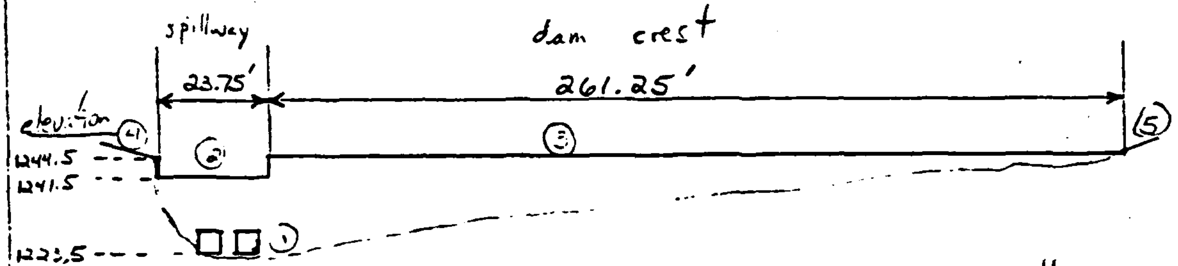
To reflect the associated hazard (middle-low) the Test Flood is chosen to be:

Test Flood = 1800 cfs


Pinnacle
West Canaan
Canaan
BOSTON
AND
MAINE
Mirror Lake
Haley Sch.
Jones Hill Sch.
Lockehaven
Height of Land Sch.
East Hill Sch.
Spectacle Pond
Crystal Lake
Isinglass Mt
Enfield Center
Montcalm Hill
MICA MINE
Fish Market
Prospect Hill
George Pond
Canaan Pond
Ford Hill
Banks Pinnacle
Halfmoon Pd
Cole Pd
Butternut Pond
George Hill Sch.
Prescott Hill
Hale Pd
GRAFTON CO
SULLIVAN CO
Aaron Ledge
Shaw Hill
SPRINGFIELD
SPRI
D-4



equivalent dam



Flow

Q① = sluice gate flow (two 3'x2' gates)

Q② = spillway flow (width 23.75')

Q③ = dam crest flow (width 261.25')

Q④ = flow over side slopes (slope = 5:1)

Q⑤ = flow over side slopes (slope = 10:1)

Assume h = 0 at the spillway crest; for all the following calculations the water level is assumed to be at the spillway crest. This is to check the dams overtopping potential with the worst prior conditions.

for $h<0$

$$Q_1 = 0.57(12.0\ ft^2)\sqrt{2(32.2)(h+18)}$$ [1] with both gates open

$$Q_2 = Q_3 = Q_4 = Q_5 = 0$$

for $0<h<3.0'$

$$Q_1 = 0.57\ (12.0\ ft^2)\sqrt{2(32.2)(h+18)}$$

$$Q_2 = 3.0\ (23.75')\ h^{3/2}$$

$$Q_3 = Q_4 = Q_5 = 0$$

for $h>3.0'$

$$Q_1 = 0.57\ (12)\sqrt{2(32.2)(h+18.0)}$$

$$Q_2 = 3.0\ (23.75)\ h^{3/2}$$

$$Q_3 = 3.0\ (261.25)(h-3.0)^{3/2}$$

$$Q_4 = 2.8\ (5(h-3.0))(0.5(h-3.0))^{3/2}$$

$$Q_5 = 2.8\ (10(h-3.0))(0.5(h-3.0))^{3/2}$$

The listing of an output from a program to calculate the head-discharge relationship follow.

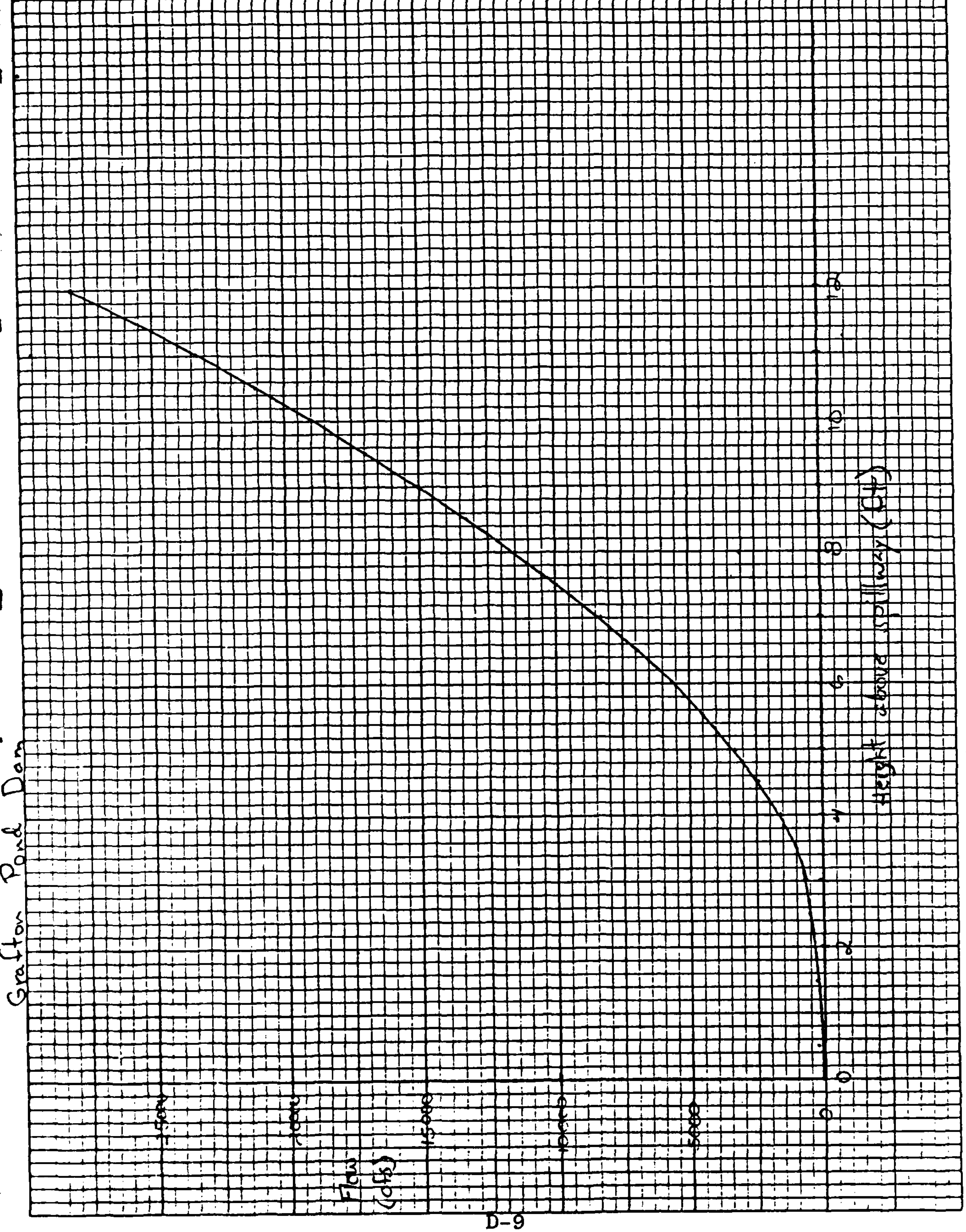
The maximum capacity of the sluice gates and the spillway before overtopping is 622 cfs from the computer output. The data has this flow calculated to be 684 cfs a 9% difference. The computer results are more conservative

1. Underflow sluice gate equation, Rouse Engineering Hydraulics, p.CC

```
100 REMARK: DISCHARGE CALCULATION FOR GRAFTON POND - GATE OPEN
110 PAGE
120 E=1.5
130 PRINT "DISCHARGE FROM GRAFTON POND - GATE OPEN"
140 PRINT USING 150:
150 IMAGE // 2T"HEAD"30T"DISCHARGE"
160 PRINT USING 170:
170 IMAGE 1T"(FEET)"32T"(CFS)"
180 PRINT USING 190:
190 IMAGE 10T"TOTAL    GATES   SPILLWAY   DAM CREST   SIDE SLOPES"
200 REMARK:      Q7 is flow over the side slopes, Q3 is flow over the
210 REMARK: dam crest, Q2 is flow over the spillway, and Q1 is flow
220 REMARK: through the gate.
230 FOR H=0 TO 13.5 STEP 0.5
240 Q1=0.57*12*(2*32.2*(H+18))↑0.5
250 Q2=3*23.75*H↑E
260 Q3=0
270 Q4=0
280 Q5=0
290 IF H<=3 THEN 320
300 Q4=2.8*(5*(H-3))*(0.5*(H-3))↑E
305 Q5=2.8*(10*(H-3))*(0.5*(H-3))↑E
310 Q3=3*261.25*(H-3)↑E
320 Q7=Q4+Q5
330 Q6=Q1+Q2+Q3+Q7
340 PRINT USING 350:H,Q6,Q1,Q2,Q3,Q7
350 IMAGE 1T,2D.2D,9D,8D,10D,11D,13D
360 NEXT H
370 END
```

DISCHARGE FROM GRAFTON POND - GATE OPEN

HEAD (FEET)	DISCHARGE (CFS) TOTAL	GATES	SPILLWAY	DAM CREST	SIDE SLOPES
0.00	233	233	0	0	0
0.50	261	236	25	0	0
1.00	311	239	71	0	0
1.50	373	242	131	0	0
2.00	447	245	202	0	0
2.50	530	249	282	0	0
3.00	622	252	370	0	0
3.50	1001	255	467	277	3
4.00	1626	257	570	784	15
4.50	2421	260	680	1440	41
5.00	3361	263	797	2217	84
5.50	4430	266	919	3098	147
6.00	5620	269	1047	4072	231
6.50	6925	272	1181	5132	340
7.00	8339	274	1320	6270	475
7.50	9860	277	1463	7482	638
8.00	11485	280	1612	8763	830
8.50	13211	283	1766	10109	1053
9.00	15037	285	1924	11519	1309
9.50	16962	288	2086	12988	1600
10.00	18984	290	2253	14515	1925
10.50	21103	293	2424	16098	2287
11.00	23317	296	2599	17734	2688
11.50	25627	298	2779	19423	3128
12.00	28032	301	2962	21161	3608
12.50	30532	303	3149	22949	4131
13.00	33125	306	3340	24784	4696
13.50	35813	308	3534	26666	5305


Grafton Pond Dam
Height above spillway (ft)
0
2
4
6
8
10
12
Flow (cfs)
0
5000
10000
15000

Grafton Pond

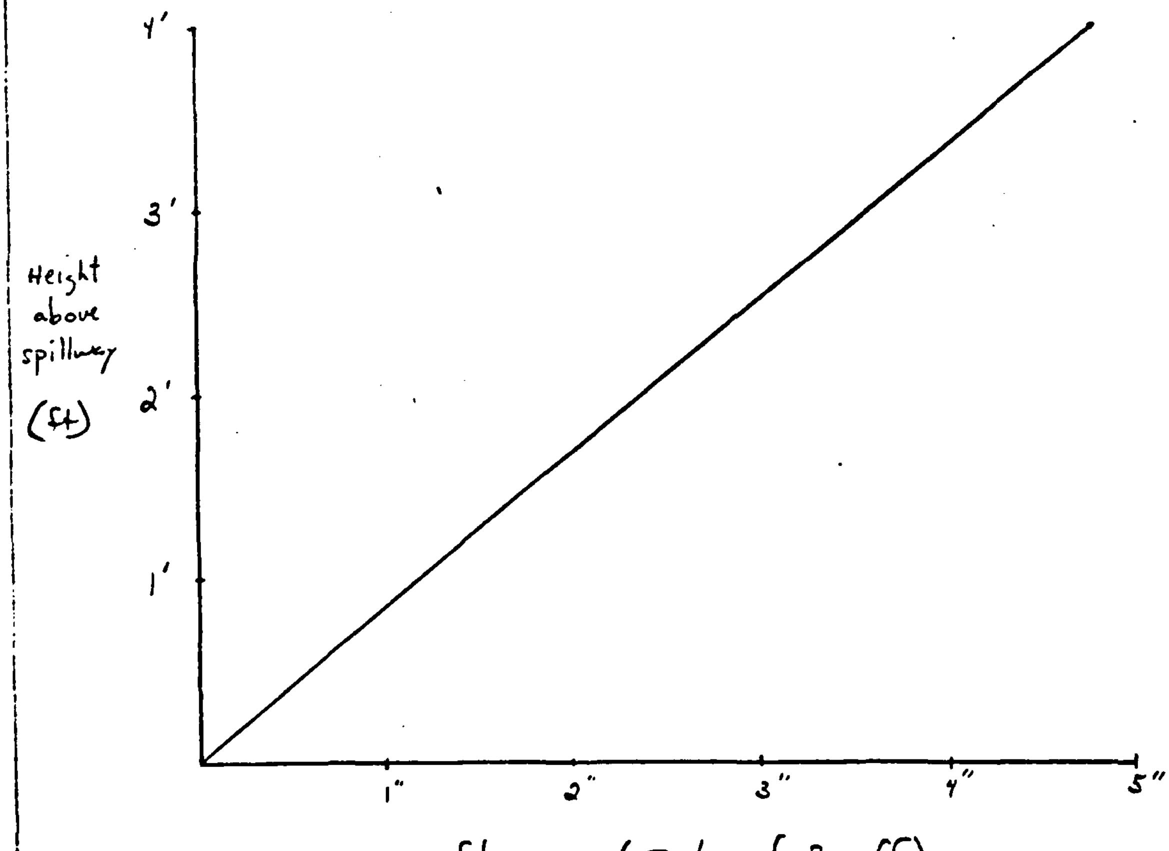
Storage - Stage Relationship :

Surface Area of Pond at normal elevation = 231 acres
= 0.361 mi^2

1" of runoff yields → $\frac{1"(3.6\ mi^2\,(640\ ac/sq.mi)}{231\ acres}$

→ 9.974" rise in water surface

1' of rise results from $\frac{12}{9.974}$ = 1.20" of runoff.




Storage Above Spillway
20
16
12
8
4
16
14
12
10
8
6
4
2
Height above Spillway Ft
3850
3080
2310
1540
770
Storage Above Spillway (Ac-Ft)

Grafton Pond

Reduction in Flow due to Storage:

Assume total storm volume = 8"

Then use COE suggested methodology with additional iterations:

$$Q_{P_2} = Q_{P_1}\left(1 - \frac{STOR1}{18}\right)$$

① $Q_{P_1} = 1800$ cfs $\longrightarrow$ $H_1 = 4.11$ feet

$H_1 = 4.11$ feet yields $\rightarrow$ $4.11(1.20) = 4.93$ inches surcharge storage

$\therefore$ $Q_{P_2} = 1800\left(1 - \frac{4.93}{8}\right) = 690.5$ cfs

② $Q_{P_2} = 690.5$ cfs $\longrightarrow$ $H_1 = 3.09$ ft

$H_1 = 3.09$ feet yields $\rightarrow$ $3.09(1.20) = 3.71$ inches surcharge storage

$\therefore$ $Q_{P_3} = 1800\left(1 - \frac{3.71}{8}\right) = 965.6$ cfs

③ $Q_{P_3} = 965.5$ cfs $\longrightarrow$ $H_1 = 3.45$ feet

3.45 ft yields $\rightarrow$ $3.45(1.20) = 4.14$ inches surcharge storage

$\therefore$ $Q_{P_4} = 1800\left(1 - \frac{4.14}{8}\right) = 868.5$ cfs

④ $Q_{P_4} = 868.5$ cfs $\longrightarrow$ $H_1 = 3.33$ feet

3.33 ft yield $\rightarrow$ $3.33(1.20) = 3.99$ inches surcharge storage

$\therefore$ $Q_{P_5} = 1800\left(1 - \frac{3.99}{8}\right) = 902.2$ cfs

⑤ $Q_{P_5} = 902.5$ cfs $\longrightarrow$ $H_1 = 3.37$ feet

3.37 feet $\rightarrow$ yield $3.37(1.20) = 4.04$ inches surcharge

⑥ Final Q_P $\rightarrow$ avg surcharge storage = $(4.04 + 3.99)/2 = 4.02$ in

$\therefore$ $Q_{P_5} = 1800\left(1 - \frac{4.02}{8}\right) = 896.2$ cfs

So H = <u>3.36 feet</u> $\rightarrow$ dam overtopped by <u>0.36 feet</u>

Grafton Pond

Calculation of Estimated Downstream Dam Failure Flood Stages - Based on COE "Rule of Thumb" Guidelines, April 1978.

Step 1: Reservoir Storage at Time of Failure

First assume the failure occurs when the dam is overtopped (USGS elev. = 1244.5')

Storage = Normal + Surcharge = 2300 + (3)(231) = 2993 Ac-ft

Step 2: Peak Failure Outflow

$Q_p = 8/27\ W_b \sqrt{g}\ Y_o^{3/2}$

W_b = breach width < 40% width = 114.0'
W_b = 100'

Y_o = depth of dam = 1244.5 - 1223.5 = 21.0'

$Q_p = 8/27\,(100)\sqrt{32.2}\,(21)^{3/2}$

= 16,180 cfs

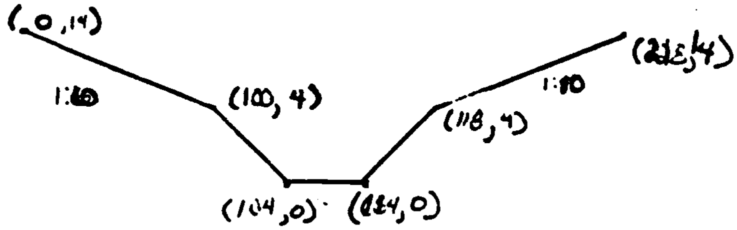
Step 3: Develop Stage - Discharge Routing for Downstream Reaches (Cross-sections from USGS topo maps and field data)

There are very few structures downstream; just three bridges. These bridges are small and are assumed to fail when and if the dam fails. The downstream floodway from Grafton Pond Dam is divided into two Reaches. These reaches are shown on the following page.

Grafton Pond

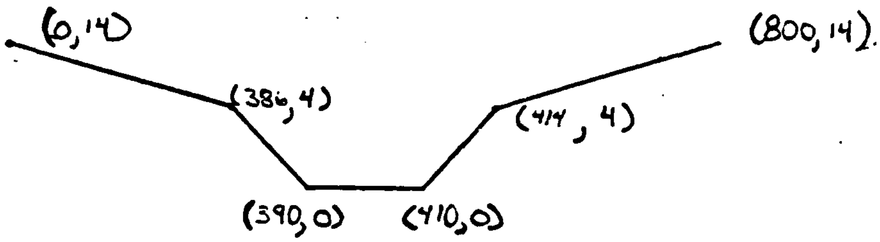
Reach 1: From Dam to where slope changes

$L = 3000'$ $n = 0.04$ $S = \frac{1223.5 - 1140}{3000'} = 0.028$

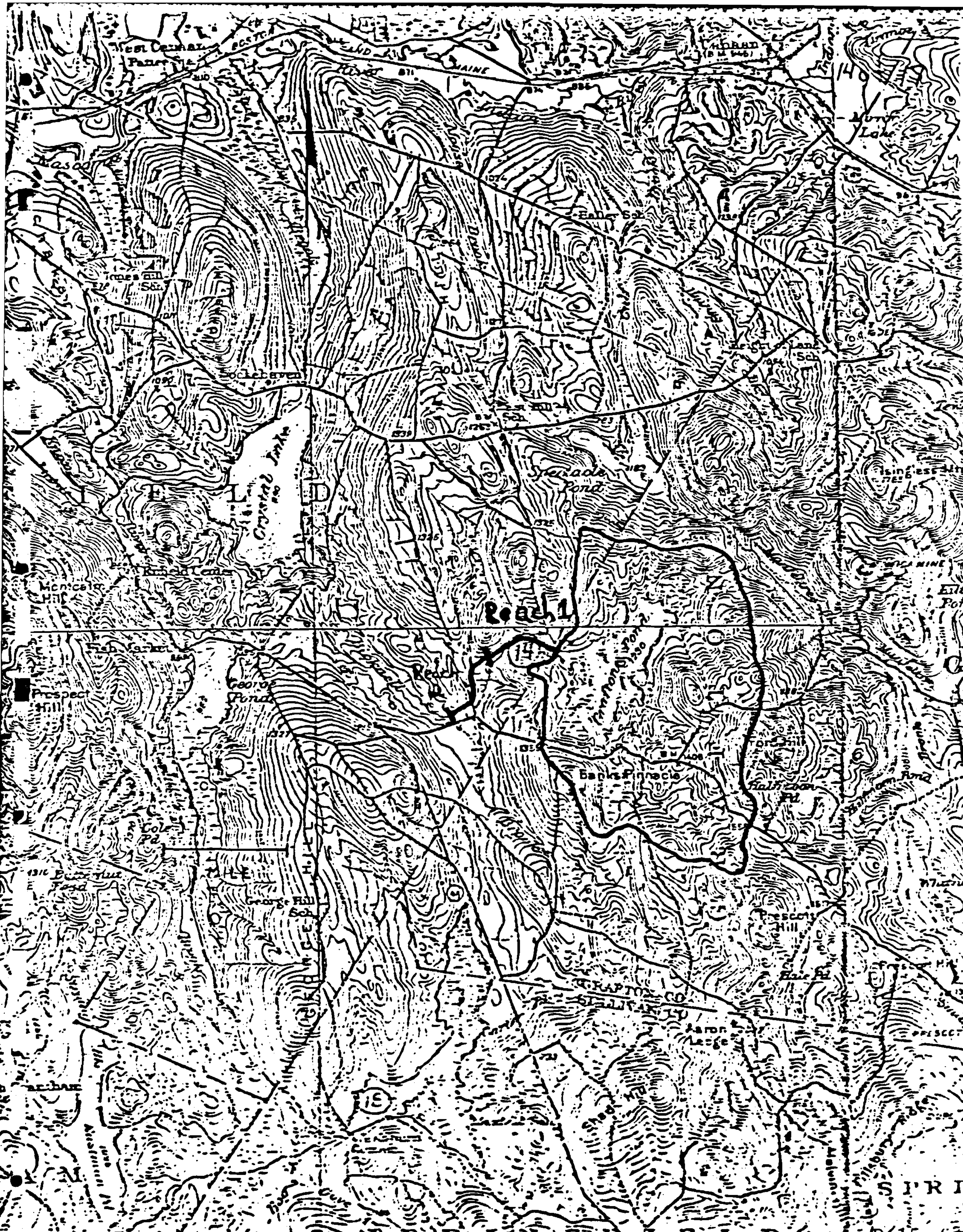


Reach 2: From first reach to Bicknell Brook

$L = 3700'$ $n = 0.04$ $S = \frac{1140-1090}{3500} = 0.014$



The following pages contain computer output tables of Stage - Discharge Relationships as well as a map showing the two reaches.


Reach 1
Reach 2
Crystal Lake
George Pond
Spectacle Pond
Banks Pinnacle
Prescott Hill
Grafton Co
Sullivan Co
Prospect Hill
Cole Pd.
Butternut Pond
George Hill Sch.
Eastman Pond
Aaron Ledge
Halfmoon Pd.
S P R I N G F I E L D
S P R I
D-15

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.5	0.5	5.3	11.4	0.5	3.1	19.5
1.0	1.0	11.0	12.8	0.9	9.9	61.9
1.5	1.5	17.3	14.2	1.2	19.6	122.2
2.0	2.0	24.0	15.7	1.5	31.9	198.9
2.5	2.5	31.3	17.1	1.8	46.8	291.5
3.0	3.0	39.0	18.5	2.1	64.2	400.0
3.5	3.5	47.3	19.9	2.4	84.1	524.3
4.0	4.0	56.0	21.3	2.6	106.7	664.8
4.5	4.5	67.5	31.4	2.2	112.5	701.5
5.0	5.0	84.0	41.4	2.0	134.6	839.2
5.5	5.5	105.5	51.5	2.1	170.3	1061.4
6.0	6.0	132.0	61.5	2.1	219.7	1369.2
6.5	6.5	163.5	71.6	2.3	283.7	1768.3
7.0	7.0	200.0	81.6	2.5	363.6	2266.6
7.5	7.5	241.5	91.7	2.6	460.8	2872.4
8.0	8.0	288.0	101.7	2.8	576.6	3594.1
8.5	8.5	339.5	111.8	3.0	712.4	4440.2
9.0	9.0	396.0	121.8	3.3	869.4	5418.8
9.5	9.5	457.5	131.9	3.5	1048.9	6538.2
10.0	10.0	524.0	141.9	3.7	1252.4	7806.1
10.5	10.5	595.5	152.0	3.9	1480.8	9230.3
11.0	11.0	672.0	162.0	4.1	1735.6	10818.4
11.5	11.5	753.5	172.1	4.4	2017.9	12577.7
12.0	12.0	840.0	182.1	4.6	2328.8	14515.5
12.5	12.5	931.5	192.2	4.8	2669.5	16639.0
13.0	13.0	1028.0	202.2	5.1	3041.0	18955.1
13.5	13.5	1129.5	212.3	5.3	3444.6	21470.6
14.0	14.0	1236.0	222.3	5.6	3881.2	24192.3

GRAFTON POND REACH ONE

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.5	0.5	10.3	21.4	0.5	6.3	27.6
1.0	1.0	21.0	22.8	0.9	19.9	87.5
1.5	1.5	32.3	24.2	1.3	39.0	171.9
2.0	2.0	44.0	25.7	1.7	63.1	277.9
2.5	2.5	56.3	27.1	2.1	91.6	403.8
3.0	3.0	69.0	28.5	2.4	124.5	548.7
3.5	3.5	82.3	29.9	2.8	161.5	712.0
4.0	4.0	96.0	31.3	3.1	202.7	893.3
4.5	4.5	119.7	69.9	1.7	171.2	754.6
5.0	5.0	162.6	108.5	1.5	212.9	938.4
5.5	5.5	224.9	147.2	1.5	298.3	1314.9
6.0	6.0	306.4	185.8	1.6	427.8	1885.5
6.5	6.5	407.3	224.4	1.8	606.1	2671.3
7.0	7.0	527.4	263.0	2.0	838.9	3697.4
7.5	7.5	666.9	301.6	2.2	1132.1	4989.5
8.0	8.0	825.6	340.2	2.4	1491.3	6573.0
8.5	8.5	1003.7	378.8	2.6	1922.3	8472.4
9.0	9.0	1201.0	417.4	2.9	2430.3	10711.5
9.5	9.5	1417.7	456.1	3.1	3020.6	13313.5
10.0	10.0	1653.6	494.7	3.3	3698.4	16300.7
10.5	10.5	1908.9	533.3	3.6	4468.5	19695.0
11.0	11.0	2183.4	571.9	3.8	5335.9	23517.7
11.5	11.5	2477.3	610.5	4.1	6305.1	27789.6
12.0	12.0	2790.4	649.1	4.3	7380.8	32530.8
12.5	12.5	3122.9	687.7	4.5	8567.6	37761.4
13.0	13.0	3474.6	726.3	4.8	9869.7	43500.6
13.5	13.5	3845.7	765.0	5.0	11291.7	49767.7
14.0	14.0	4236.0	803.6	5.3	12837.6	56581.4

GRAFTON POND REACH TWO

Grafton Pond

Step 4: Calculate Downstream Attenuation

Reach 1 - $Q_{P_1} = 16180$ cfs $\rightarrow h_1 = 12.39$ feet
$\rightarrow A_1 = 911.72\ ft^2$

$$Volume_1 = \frac{(3000' \times 911.72\ ft^2)}{43560} = 62.79\ Ac\text{-}ft$$

$$Q_{P_2T} = 16180\left(1 - \frac{62.79}{2993}\right) = 15840\ cfs$$

15840 cfs $\rightarrow H_2 = 12.31$ ft
$\rightarrow A_2 = 897.10\ ft^2$

$$V_2 = \frac{3000(897.1)}{43560} = 61.78$$

$$V_{AVG} = \frac{62.79 + 61.78}{2} = 62.28\ Ac\text{-}ft$$

$$Q_{PAVG} = 16180\left(1 - \frac{62.28}{2993}\right) = 15843\ cfs \rightarrow h_2 = 12.31\ ft$$

Reach 2 - $Q_{P_1} = 15843$ cfs $\rightarrow h_1 = 9.92$ ft
$\rightarrow A_1 = 1617.5\ ft^2$

$$V_1 = \frac{3700(1617.5)}{43560} = 137.4\ Ac\text{-}ft$$

$$Q_{P_2T} = 15843\left(1 - \frac{137.4}{2993}\right) = 15116\ cfs$$

15116 cfs $\rightarrow h_2 = 9.80$ ft
$A_2 = 1560.0\ ft^2$

$$V_2 = \frac{3700(1560)}{43560} = 132.5\ Ac\text{-}ft$$

$$V_{AVG} = \frac{137.4 + 132.5}{2} = 135.0\ Ac\text{-}ft$$

$$Q_{P_2} = 15843\left(1 - \frac{135.0}{2993}\right) = 15128\ cfs \rightarrow h = 9.80\ ft$$

Reach	Stage	Discharge to downstream
1	12.31 ft	15843 cfs
2	9.80 ft	15128 cfs

APPENDIX E

INFORMATION AS CONTAINED IN

THE NATIONAL INVENTORY OF DAMS

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION		STATE	COUNTY	CONGR. DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
NH	11? [illegible]	[illegible]	[illegible]				GRAFTON POND DAM	4334.8	7202.7	27NOV78

POPULAR NAME	NAME OF IMPOUNDMENT
	GRAFTON POND

[illegible]	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI.)	POPULATION
[illegible]	TR-BICKNELL BROOK	LOCKEHAVEN	4	150

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	NORMAL (ACRE-FT.)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
[illegible]	1918	RC	21	21	3000	2300	NED	N	N	N	N	13DEC78

REMARKS
21-CONCRETE AMBURSEN TYPE 23-FORMERLY HYDROELECTRIC STORAGE

SPILLWAY [illegible]	TYPE	WIDTH	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	PROPOSED (MW)	NO	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
235		24	370												

NAVIGATION LOCKS

OWNER	ENGINEERING BY	CONSTRUCTION BY
[illegible] RES BD	MASCOMA RIVER IMPROV CO	MASCOMA RIVER IMPROV CO

REGULATORY AGENCY

DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
[illegible]	NONE	NONE	NONE

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
[illegible] ZOINO DUNNICLIFF + ASSOC	20SEP78	PUBLIC LAW 92-367

REMARKS
[illegible] FORMERLY GRANITE STATE ELECTRIC CO

END

FILMED

8-85

DTIC